THE PREDICTION OF STRUCTURAL FAILURES.

A thesis submitted to the University of London for the degree of Doctor of Philosophy.

BY

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Frontispiece.

The Prediction of Structural Failures

Abstract

This thesis is mainly concerned with the prediction of one class of structural failure, namely that due to the extrapolation of existing design or construction procedures to fit new situations.

Four failures were examined in depth in an attempt to establish a realistic procedure for avoiding such accidents. The structures studied were large metal bridges because these have represented the forefront of technology for as long or longer than any comparable example. The examples chosen were the Dee Railway Bridge (collapsed 1847), the Tay Bridge (1879), the first Quebec cantilever bridge (1907) and the Tacoma Narrows suspension bridge (1940). Great care was taken to study the accidents and preceding events as contemporary engineers would have seen them, and it is believed that this is the first time this type of investigation has been made.

The pattern that emerged from this was that the designers all used existing data which they were confident applied to their work. Not until after the accident did it become apparent that this was not the case, and that the data had originally been derived with very different conditions in mind, often for applications which seemed insignificant alongside the failed structure.

In conclusion the author suggests how similar situations could be avoided in the future by the setting up of review procedures to assess new developments in the light of existing practices and the facts on which these are based. An outline of the costs of the accidents studied suggests very strongly that such work would be economically justifiable.

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The Prediction of Structural Failures

1 Introduction

1.1 The Study of Structural Safety

This thesis is concerned with one very important aspect of the general study of structural safety, namely the prediction of design situations in which disasters are likely to occur. Since the main subject matter only covers a small part of the overall field of safety in civil engineering, it is as well to begin by drawing a general picture of work that has been done on all aspects of the subject, with a view to putting the present study into perspective.

To start with a definition, to be considered safe a structure has to be capable of sustaining all reasonable loads applied to it throughout its design life without collapsing, without exhibiting undue deflections and without requiring excessive maintenance. This suggests two possible ways of studying the subject; either a safety philosophy is applied to the design process to guarantee that new structures will stand up; or, under one guise or another, measures are devised to make sure they will not fall down. Both methods have advantages and disadvantages, as can be shown by an outline of work accomplished during the last 25 years.

The need for a coherent safety philosophy was first put to the Civil Engineering profession by A G Pugsley in a paper written in 1951.[1] In this paper, the author pointed out that it was anomalous to continue using the traditional, but largely empirical 'safety factors' in conjunction with the increasingly sophisticated techniques of modern analysis. He envisaged several possible developments all of which he and others have worked on since.

One of the measures he advocated was a return to factors of safety based on loads (i.e. the ratio of working to collapse loads). A method of calculating appropriate safety margins from this criterion was outlined in the Report of the Institution of Structural Engineers Safety Committee which appeared in 1955.[2]

Here it was suggested that the uncertainty of the various aspects of the design process (such as predicted loading, accuracy of analysis, workmanship, danger to personnel, and cost of safety measures) should each be assessed individually. A rational margin of safety could then be calculated by ascribing numerical values to each element and combining these partial safety factors by multiplication.

In 1971 a report was produced by the Safety Committee of the Construction Industry's Research & Information Association. This discussed the partial safety factor approach with particular reference to the advantages and disadvantages that had been revealed by several years application to the design of reinforced concrete and other structures.

It was found that the rationalisation of the individual safety factors was hampered by lack of information. For example, knowledge of external loads left much to be desired (except in isolated instances such as wind loading where much progress had been made), as did knowledge of the actual strength of existing structures. The only way of improving matters was seen to be by the collection of an immense amount of data for each class of structure and each type of load, a problem which was made doubly difficult by the fact that so many large civil engineering works are essentially one-off jobs.

In the future there will undoubtedly be progress with this approach, but it is bound to be slow and painstaking, and for a long while intuitive as much as rational. Pugsley, who was chairman of both committees seems to have realised this,[3] as did the recent Interim Committee on Structural Safety sponsored jointly by the Institutions of Civil, Structural and Municipal Engineers.[4]

Another safety philosophy, based on the mathematical probability of a failure resulting from an unusually weak structure being acted on by an abnormally high load is also restricted in application at the present time by a lack of loading and strength data, strength data in particular being virtually unobtainable without tests to destruction (impracticable for most civil engineering structures) or a return to proof loading acceptance tests.

Both these approaches suffer from real problems relating to the human user. In the partial load factor case a risk of danger to personnel which may seem reasonable to the designer because it is parallel to other risks the user takes may in fact be unacceptable. Pugsley, for example, has pointed out that travellers on a Motorway expect a much greater level of safety from the bridges they cross than from the cars they travel in.[5] This was recently demonstrated, when, during the £2½m strengthening of Tinsley Viaduct in Sheffield to meet the requirements of the Merrison box girder rules a lorry ran out of control and through the bridge parapet because no crash barrier (cost £30,000 per mile) had been installed. In the mathematical strength/load treatment a clearly defined probability of failure has to be designed for, and this has also proved unacceptable to the public cognoscenti.

In the light of these remarks there is clearly scope for work based on human parameters. The public, for example, welcomes direct attempts to restrict accidents because that is its main concern, and indeed safety rules such as the International Airworthiness Regulations for aircraft are based on actual accident rates. In the author's opinion, conscious effort aimed at minimising the incidence of accidents is a necessary complement to work on the general philosophy of safety, and it is this aspect of the field that was selected for study in this thesis.

The most obvious way of endeavouring to reduce the numbers of accidents is by examining past failures and seeing what, if anything, they have had in common. With this aim in mind several other writers have made collections of accident case histories, but usually the field covered and the conclusions drawn have been so general as to be bewildering to the reader. Works that fall into this category include Stamm's "Bruckeneinsturze und Ihres Lehren" (1952), Rolt Hammond's "Engineering Structural Failures"(1956), McKaig's "Building Failures"(1962), Feld's "Construction Failures"(1968), the Institution of Civil Engineer's report on "Safety in Engineering"(1969) and Scott's "Building Disasters and Failures"(1976). All of these proceed by a recitation of the circumstances surrounding many collapses and the mistakes to be avoided in the future. This is unsatisfactory because it results in such a vast check list of points to be considered that a designer who consults and follows it may be lulled into a sense of security that dulls his fundamental thinking about safety matters.

As a result one aim of this thesis is to discipline the study of failures, or at least some part of it, so that it helps designers in a positive rather than a negative way. This is done by using the records of previous accidents to predict the possibility of future collapses rather than to eulogise on past catastrophes. With respect to accident case histories this means looking for patterns of events rather than at each collapse individually.

1.2 Types of Accident

A sensible starting point for the study of collapses is to classify what can usefully be studied.

Structural accidents, it seems, fall into three categories of which two can usefully be researched. This thesis is mainly concerned with the possibility of avoiding one of the types, but it is as well to begin by setting out all three categories.

Firstly, there are accidents due to causes such as simple calculation errors or straightforward mistakes on site. This class of failure includes any incident whose cause is immediately obvious to an average engineer reviewing the facts. Most accidents fall into this category, but so long as human beings are fallible there is probably not much that can be done to prevent them.

Next there are accidents due to causes such as earthquakes, floods or hurricanes which are traditionally looked on as acts of God but which in fact can be guarded against by the collection of sufficient loading data. Knowledge of the probability of a certain loading being exceeded enables a designer to take a calculated risk in his specification of a new structure. The problem of avoiding this type of accident or of keeping the number of instances down to an acceptable level is properly the preserve of the scientists specialising in the collection and analysis of the loading information.

The third type of accident is the smallest category but is also the most significant because relatively simple measures could be applied to largely prevent the occurrence of similar disaster situations in the future.

In this class of failure, the distinguishing feature of the structures is that although the design complies with the accepted practice of the day, the structure is intriniscally incapable of sustaining the loads applied to it. The primary cause of such accidents is frequently found to be the use of a design method, construction technique or material unwittingly applied outside its range of validity - usually some principle has been developed and then used for ever larger applications until finally it is mistakenly employed for a structure of such scale and geometric proportions that a fundamentally different approach is required. By studying the history of certain structures which have failed in the past it is possible to detect a repeating pattern of events which could be broken if certain measures were taken to break dangerous cycles of development.

1.3 Area of Study

The subject area selected for this thesis was the study of this type of accident as illustrated by the history of large metal bridges. This class of structure was selected because it has always represented the vanguard of a particular area of structural history, namely the application of iron to building purposes. Just as the cast-iron arch at Ironbridge in the Severn Gorge was the first important load bearing structure to be made of metal (1779), so the Dee Railway Bridge at Chester (opened 1846, collapsed 1847) was the longest cast iron trussed girder, earlier applications having been confined to factories (under the guise of fireproof buildings), or shorter spans over or under railways.

During the age of wrought iron (roughly 1850-1885) the huge spans of the Britannia (1849), Saltash (1859), Clifton (1863) and Garabit (1885) bridges were only rivalled as construction projects by the mighty Tour Eiffel in Paris (1889, height 300m), the St Pancras Station roof (1870 span 70m length 210m) and by ships such as Brunel's Great Britain and Great Eastern. Another structure of great daring dating from this period was Sir Thomas Bouch's sinuous Tay Bridge (opened 1878, collapsed the following year), stretching for two miles across one of the roughest and most exposed estuaries in the British Isles and representing an attempt to combat extreme loading conditions with a structure of ruthless economy.

In the years following the introduction of steel the great Forth Bridge (1889) with main spans of 530m and towers rising 102m from their foundations was a symbol of achievement challenged only by the New York skyscrapers and the ill-fated Quebec Bridge (collapsed during construction in 1907).

During the twentieth century there has been an explosion in the dimensions of all sorts of structures. The world's tallest building is now 444m high (Sears Tower Chicago) and steel masts stretch over 600m into the sky. In the field of bridging the greatest span achieved by the suspension form now favoured for all long unobstructed crossings has risen from 486m (Brooklyn 1883) to 1410m (Humber Bridge, due for completion in 1977). During this period one exceptionally daring if not outstandingly long structure collapsed. This was the Tacoma Narrows Bridge which failed in 1940.

The four accidents just mentioned make up the body of this thesis because they all fit the criterion that defines the class of failure selected for study. In each case the structure which collapsed was preceded by the seemingly logical development and extrapolation of a design principle. Disaster resulted when changes made in the parameters involved led to the structure behaving, not in the way earlier examples had done, but in a new and unpredicted fashion.

1.4. Method of Study

The principal novelty of this thesis lies in detecting and demonstrating that this pattern exists and that it repeats itself from time to time. The study looks beyond the superficial details of what happened when a structure collapsed and examines the processes that led up to the disaster, because it is there that the fundamental similarities between the accidents are to be found.

Although the thesis is only concerned with one pattern and one area of the whole construction process, the author believes that it is typical of a number of similar patterns that could be isolated in fields such as the use of materials and the application of certain fabrication techniques. The use of cast iron, firstly for arches but later in beams is an example of the first topic, while the failure of welded bridges in Belgium and Germany in the 1930s illustrates the latter.

This study is the first, so far as the author knows, to investigate accidents in this way. In addition to establishing that the four failures described were the result of a similar approach to the design problem, the author shows that certain measures could have been taken to prevent the design cycle ending in disaster.

The most important of these is to suggest that the continuous collection and publication of data in parametric form relating to the structures in a particular class would help a designer assess the basic degree of risk taken in developing some aspect of the design. This applies particularly to structures where there is a reasonable degree of uniformity, and only to a lesser extent to those of great individuality. The studies suggest that the designers of the structures discussed herein did not appreciate the size of the step forward they were taking, and that had they done so they would have made proper recourse to experimentation.

Another measure discussed provides clues that a disaster situation was imminent in each case. This, rather surprisingly, is the unusual fact that each accident was preceded by a number of similar incidents on a smaller scale which, although recorded, were not wholly understood by the engineers of the day. This suggests that it is worthwhile to keep a register of all structural failures where the cause cannot be wholly explained in the hope that this would highlight uncertainties and stimulate research where it is required. The importance of keeping this accident history file separate from one covering the general run of collapses cannot be over-emphasised.

1.5 Earlier Studies

Having outlined the method of study used in this thesis, it is worthwhile reviewing the work of earlier writers who have discussed the four bridge accidents.

In each case the first investigation was carried out under some form of Government Commission supported in one of a number of ways, but always with sufficient legal power to investigate all the circumstances of the collapse.

The Dee Bridge failure of 1847 was the subject of a dramatic Coroner's Inquest, nominally to determine the cause of death of those unfortunate enough to be killed in the accident but in fact more concerned with determining the cause of the collapse since at that time there was no established way of investigating a structural failure.[6] The Jury returned a strongly worded verdict calling for an inquiry into the application of iron to railway structures, which materialised as a Royal Commission shortly afterwards.[7] The Commissioners, under the chairmanship of Lord Wrottesley, concentrated on solving the problems isolated at the Inquest relating to uncertainties in the use of iron for railway bridges. These included the resistance of structures to static forces and the provision required for impact and rolling loads. They also compared the different types of bridge that could be used for railways and included a long appendix on the then fashionable tubular structures. Their report also drew attention to uncertainties revealed in the course of their inquiries, including matters such as the precise physical properties of cast and wrought iron.

There was, however, no attempt at understanding why the original accident happened, when and how it did, which is what the present study sets out to do. Much the same can be said about the Report of the Tay Bridge Inquiry (in fact the Commissioners produced two reports more or less parallel in content except that one saw fit to apportion blame for the accident onto individuals.)[8] Here the Commissioners examined the principal witnesses more or less in the manner of a criminal trial, raking over the many malpractices that passed unchecked during the construction of the bridge, and partially eclipsing the true cause of the accident which was insufficient provision for wind forces. Having drawn attention to the shameful workmanship and indifferent project management, and having isolated the wind-loading problem as requiring further research they considered their task completed. They made no study of how it had been possible for Thomas Bouch to design and construct an inadequate structure for the longest crossing in the world in full view of the whole engineering profession. This is now attempted for the first time.

Whereas the Tay Bridge Inquiry was beset with the very unpleasant and hostile atmosphere of a courtroom, the Quebec Bridge investigation resulted in an extremely well-balanced and impartial report.[9] Here the Commissioners devoted some attention to tracing the history of the data which was used in the design with such disastrous consequences. This in the Commissioners' view merely showed that the background of research and experiment was inadequate, but their work is here extended to show how an ongoing historical study could have enabled the accident situation to be foreseen.

The destruction of the Tacoma Narrows Bridge was investigated along similar lines although the minutes of evidence were never published in full. In the case of this , which was the last bridge to be studied, the investigators made a sketchy attempt to put the collapse into perspective in a manner quite similar to that adopted by the present author, although this was unknown to him while carrying out work on the other studies. However, the Commissioners made no real attempt to assess precisely what the use of the historical material might be in preventing similar situations arising in the future, so the originality of this particular case history lies in expanding their material to do just that by referring to additional sources. The Tacoma Bridge Commissioners did not, of course, draw parallels with the other three failures.

Much time has elapsed since these accidents and the last three have featured time and again in literature about bridges. However among the general books, of which Tyrell's"History of Bridge Engineering"(1911), Steinman and Watson's "Bridges and their Builders"(1941), Gies' "Bridges and Men" (1964) and Hopkins' "A Span of Bridges"(1971) are a fair cross-section, the accidents are just treated as isolated events in the general chronology of bridge building except in the case of the Tacoma Narrows Bridge where one or two of the earlier aerodynamic collapses are sometimes mentioned.

1.6 Source Material

The study of accidents, as this thesis shows, deserves to go deeper than this. Just as the building of the great triumphs of engineering merits the step by step retelling of all the stages of design and construction, so understanding and prevention of accidents demands that the history of how the engineers of the day thought and acted, as they made the decisions that ended in tragedy, be recorded. The manner of studying accidents herein presented takes as its starting point the premise that people of different generations react to certain 'climates' and conditions in more or less the same way. It is therefore necessary to try and make the characters of people long dead and the circumstances in which they work come alive again. Much time has been spent in reading original writings and discussions, in studying old journals and periodicals, textbooks and manuscripts. This work has been greatly facilitated by ready access to the British Transport Record Office, and the libraries of the British Museum and the Institution of Civil Engineers.

As the studies were built up so the life of the engineers of history became real and not merely that of hard work justly rewarded (as portrayed by the famous biographer Samuel Smiles) nor of folly crowned with disaster. On the contrary, our forbears emerged as genuinely human, with human talents, ambitions, pride, jealousies and weaknesses just like ourselves.

The writer who inspired the present author's interest in the human aspect of engineering history was the late L T C Rolt who was the first biographer to make the great engineers of the past live again as people. Mention should also be made of a book by P S A Berridge entitled "The Girder Bridge" (published in 1969) which discusses the Dee Bridge failure and its aftermath in the words of the people involved at the time. Although Berridge wrote before the recent collapses of box girder bridges at Yarm and Milford Haven, his book shows the astonishing similarity of professional and public reaction one hundred and twenty years apart. In fact, it was the present author's reading of Berridge's discussion of the Dee Bridge and the Report of the West Gate Bridge Royal Commission simultaneously that really stimulated his interest in looking for the pattern now presented.

2. The Dee Bridge

Contents:

2.1 Introduction

The River Dee at Chester is an unexceptional river between 60 and 90m (200-300 feet) wide, with a tidal range of 3.6m (12 ieet). The waterway was spanned in 1832 when the great masonry arch of the Grosvenor Bridge strode across the river with an unprecedented leap of 61m (200 feet), so, in the early 1840s the directors of the new Chester & Holyhead Railway Company did not foresee the crossing as presenting an obstacle in any way comparable to the proposed sea wall traverse at Penmaenmawr, or the crossings of the Menai Straits & Conway estuary which were to enable their project to complete the line of railway between London and the port for Ireland.

In 1844, when the directors felt that the time was ripe to commence construction, they commissioned a report from James Rendel, who proposed a bridge of brick arches because clearance for shipping did not have to be provided. Put in his own words, "There is very little probability that it (the river upstream) will ever be required for shipping as there is ample space for any probable extension of the trade of the port, lower down the river, in the vicinity of the present wharves, where there is deeper water and where the canal basin is situated."[1] The bridge he sketched had 18.3m(60') spans, with a clearance of 6m (20') above high water level.

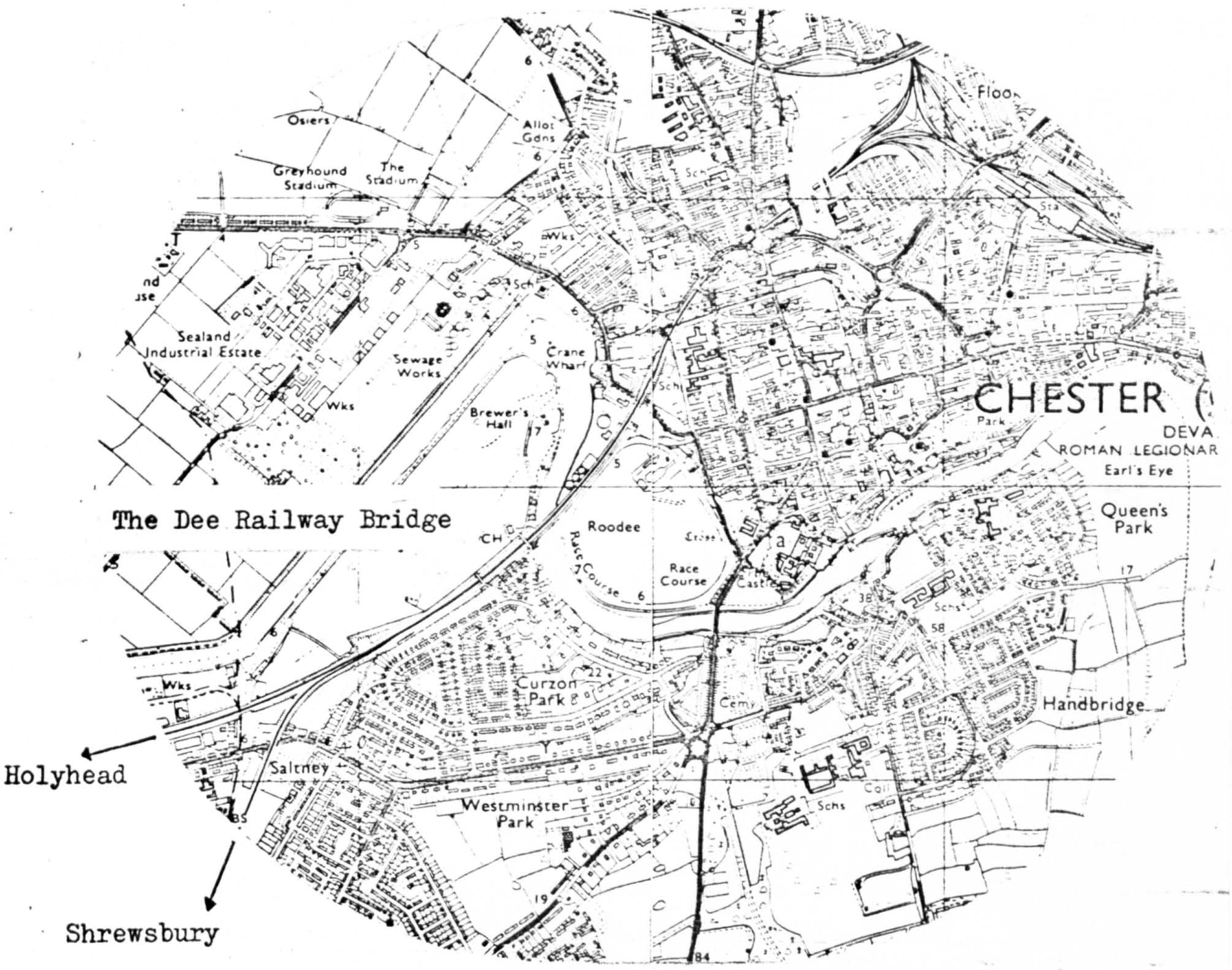
After the Parliamentary Act permitting construction of the Railway had been passed in July 1844, Robert Stephenson, newly appointed as Chief Engineer to the Company, made firm plans for crossing the Dee on a bridge of five brick arches. Very soon the first of a series of design problems along the new line of railway made itself apparent when piling in the river bed revealed poor quality foundations. Stephenson and his staff were thus faced with designing a new, lighter bridge to fit the boundary conditions imposed by the already advancing approaches, which were a cutting on the Welsh side and an arched viaduct from Chester. These demanded a 76m (250') crossing of the river on a curve 3.2 km (2 miles) radius and a basic skew of 51° (see figure 2.1).

A timber viaduct, using the piers already begun, might have been considered as an alternative under different circumstances, but here Stephenson must have felt a need to show the directors and shareholders of the Railway Company that they had indeed selected the right engineer to carry their line in a permanent and masterly fashion along the difficult route to Holyhead.

With timber and masonry excluded as structural materials the only remaining possibility was iron, which (as John Storey said in 1844)[2] bridge engineers were encouraged to build with because "it uses a material produced in this country better and cheaper than elsehwere, and assists one of its staple manufactures, which is at this moment much depressed."

The form of iron bridge which Stephenson and his staff might have considered were summarised in the "Report of the Commissioners Appointed to Inquire into the Application of Iron to Railway Structures" which was published in 1849. The Commissioners derived their information from the "Returns of Iron Bridges" made by the Railway companies in response to a Board of Trade letter circulated after the Dee Bridge collapsed.[3] (Table 2.1.)

They listed the bow and string girder, the cast iron arch , the truss bridge, the tubular bridge, the built girder and the trussed



Scale 0 I 2 km.

Fig 2.I. The site of the Dee railway bridge.

cast iron girder as possible ways of employing iron in bridges where the span was greater than that obtainable with a simple beam.[4]

In the Commissioners' Report, several leading engineers (but not Stephenson) agreed in considering the bow-string girder with a bow of cast or wrought iron cells and tension rods of wrought iron as free from any objections and urged against other modes of combining the materials. The form was first used in two bridges in Leeds built by George Leather in 1827 and 1832 respectively, (note that Whipple, who patented the idea in 1841, was not the originator of the genre). Robert Stephenson's bridge over the Grand Junction Canal at Weedon was another early example, and when the Dee Bridge fell the celebrated High Level Bridge as Newcastle was under construction (fig. 2.2). These bridges, using cast iron cells for the bow, unfortunately generated rather cumbersome and expensive structures, and it was not until wrought iron was used throughout that the form became really economical. W C Harrison's 170 foot span over the River Ouse, completed in 1848, appears to have been the first completely wrought iron example.

The cast iron arch generally proved uneconomical for iron railway bridges, because although the material was cheaper than wrought iron, any saving was offset by the need to provide extensive abutments. The problems of restricted clearance and headroom beneath the bridge could only be overcome by extensive approach embankments (fig.2.3).

Truss bridgework eventually dominated the middle range of railway spans, but in 1845 British engineers did not realise its potential.[5] The various forms shown in fig 2.4 were applied to American railways

TABLE 2.1. - page 1

RETURNS OF IRON BRIDGES RECEIVED IN RESPONSE TO A BOARD OF TRADE CIRCULAR, 1847

Ref	Name of Railway	Iron Bridges under 40' span	Iron Bridges over 40' span	Analysis of Structures over 40', with max. span (ft.)							Notes
				Cast Iron arch	Bow/ string girder	Tubular girder	Simple castiron girder	Built girder	Trussed castiron girder	Lattice truss girders	Lattice & tubular girders the only forms entirely of wrought iron.
1.	Aberdeen	4	-								
2.	Arbroath & Forfar	8	-								
3.	Buckinghamshire	9	1			1(60)					Cast iron box flange to tubular girder.
4.	Chester & Holyhead (Conway-Holyhead div.)	36	5			5(460)					Britannia & Conway tubular bridges.
5.	Birkenhead, Lancs & Cheshire Junction	3	4	4(90)							
6.	Birmingham & Oxford Junction	2	-								
7.	Blackburn Clitheroe & North West Junction	13	-								
8.	Blackburn Darwen & Bolton	25	7	4(76)		2(60)	1(50)				
9.	Chester & Birkenhead	3	-								
10.	East Lancs.	51	13	4(100)		4(62)	3(45)		2(41.5)		
11.	East Lincs	6	-								
12.	Eastern Union	10	-								
13.	Edinburgh & Glasgow	31	-								
14.	Eastern Counties	3	-								
15.	Edinburgh & Northern	28	9	1(55)				5(104)	3(69)		
16.	Ely & Huntingdon	34	2	2(50)							
17.	Fleetwood Preston & West Riding Junction	6	-								
18.	General Terminus & Glasgow Harbour	7	-								
19.	Glasgow & Paisley Joint	4	-								
20.	Glasgow, Barrhead & Neilston District	6	-								
21.	Glasgow Southern Terminal	6	3					3(46)			
22.	Glasgow Dumfries & Carlisle	7	-								
23.	Glasgow Paisley & Greenock	2	-								
24.	Glasgow Paisley Kilmarnock & Ayr	41	-								
25.	Great Northern Railway	1	1				1(48)				Brunel was among the first to abandon
26.	Great Western Railway	5	-								cast iron underbridges after 1847.
27.	Huddersfield & Manchester	-	1						1(63)		Other bridges not detailed.
28.	Lancaster & Carlisle	9	-								
29.	Lancaster & Preston Junction	12	-								
30.	Leeds Dewsbury & Manchester	9	3	3(100)							
31.	Leeds & Thirsk	8	5	5(100)							
32.	Lancs. & Yorks.	10	17			1(66)	2(42)			14(84)	
33.	London & Blackwall	33	9			2(60)	6(57)		1(65)		
34.	Lancs &Yorks branches Ardwick	2	2			2(100)					

TABLE 2.1 - page 2

RETURNS OF IRON BRIDGES RECEIVED IN RESPONSE TO A BOARD OF TRADE CIRCULAR, 1847

Ref.	Name of Railway	Iron Bridges under 40' span	Iron Bridges over 40' span	Analysis of Structures over 40', with max. span. (ft.)							Notes
				Cast Iron Arch	Bow/ string girder	Tubular girder	Simple cast iron girder	Built girder	Trussed cast iron girder	Lattice truss girder	
35.	Lancs & Yorks branches Ashton	7	-								
36.	Burnley	1	-								
37.	Wakefield Pontefract & Goole	2	10				1		9(88)		
38.	London Brighton & South Coast	36	-								
39.	Londonderry & Enniskillen	10	-								
40.	London & North Western Newton-Birmingham	5	4				4(60)				
41.	Liverpool-Manchester	9	3	3(120)							
42.	Birmingham-Manchester	18	6	5(129)					1(59)		
43.	Chester-Crewe	8	-								
44.	S. Div. branches	27	1				1(46.5)				
45.	Hampstead-Stonebridge	25	8	4(66)	2(70)	1(60)			1(64)		
46.	Rugby-Leamington	5	5			5(60)					
47.	Coventry-Nuneaton	2	4			4(60)					
48.	Lowestoft Railway & Harbour	2	-								
49.	London & South Western	3	5	5(90)							
50.	Lynn & Dereham	15	-								
51.	Manchester Sheffield & Lincs. M & S div.	15	-								
52.	Great Grimsby & S.	11	6)			2(154)			1(69)		
53.	S & Lincs.	-	3)			6(79)					Timber girders plated with w.i.
54.	Manchester & Leeds	14	7	1(63)	6(102)						
55.	Manchester South Junction & Altrincham	14	15	3(105)					12(74)		
56.	Maryport & Carlisle	3	7				7(50)				
57.	Middlesborough & Redcar	1	-								
58.	Midland 1 Erewash Valley Line	19	2				1(42)	1(52)			
59.	West Branch	30	2		2(45)						
60.	Leicester & Swannington	3	-								
61.	South Branch	33	4	4(100)							
62.	North Branch	26	2					2(50)			
63.	Bristol-Birmingham	23	1								
64.	Nottingham-Lincoln	16	-								
65.	Midland Great Western (Ireland)	3	-								
66.	Monkland	1	-								
67.	Newcastle & Berwick	9	-								
68.	Newcastle & Carlisle	9	1	1(50)							
69.	Newport & Pontypool	7	2	2(50)							
70.	Norfolk	7	2					2(45)			

TABLE 2.1 - page 3

RETURNS OF IRON BRIDGES RECEIVED IN RESPONSE TO A BOARD OF TRADE CIRCULAR, 1847

Ref.	Name of Railway	Iron Bridges under 40' span	Iron Bridges over 40' span	Analysis of Structures over 40', with max. span (ft.)						
				Cast Iron Arch	Bow/ string girder	Tubular girder	Simple cast iron girder	Built girder	Trussed cast iron girder	Lattice truss girders
71.	North British	64	1				1(41)			
72.	North Union	6	1	1(50)						
73.	Preston	2	-							
74.	St Helen's Canal	2	-							
75.	Scottish Central	2	1	1(54)						
76.	Scottish Midland	-	1	1(47)						
77.	Shrewsbury & Birmingham	11	4	3(100)					1(50)	
78.	Southampton & Dorchester	6	-							
79.	South Eastern London-Dover	39	-							
80.	Kent Branches	24	2				2			
81.	Main Line	-	-							
82.	Tunbridge Wells Branches	7	-							
83.	Stirling & Dumfries	3	-							
84.	Stockton & Darlington	9	3						3(83)	
85.	Stockton & Hartlepool	4	-							
86.	Taff Vale	4	-							
87.	Trent Valley	38	8						8(70)	
88.	Ulster	1	-							
89.	Waterford & Limerick	2	5							5(86)
90.	West Cornwall	1	-							
91.	Lancs & Yorks	6	3	1(65)		1	1			
92.	Wilsontown Morningside & Holtness	2	-							
93.	York & Newcastle	25	12					8(53)	4(60)	
94.	York & North Midland Church Fenton & Harrogate	8	3						3(69)	
95.	East Riding branches	3	5	1(90)				3(78)	1(56)	
96.	Hull & Bridlington	9	-							
97.	Seamer & Bridlington	1	1					1(46)		
98.	York & Scarborough	4	2						2(70)	
99.	Whitby & Pickering	12	4					4(42)		
100.	Dublin & Drogheda	8	6				1(43)		3(48)	2(144)
	TOTALS 1400 spans	1156	244	60	10	36	32	29	56	21

Notes

Reference: "Returns of Iron Bridges, 1847"

Public Record Office.
Manuscript Text; M.T.8
Manuscript plans; M.P.I:226,227

226 Companies asked for information:-

77 replies received
38 lines not yet built
48 lines with no iron bridges
52 no reply
11 miscellaneous (Amalgamated Coys.etc)
226

General Comments

1. In 1847 there were 3 times as many cast iron/composite cast & wrought bridges over 40' than there were wrought iron structures.
2. All bridges under 40' span were cast iron girders. Many were proportioned with scant regard for Hodgkinson's Formula.
3. There was a great diversity of practice within the general classifications.
4. There was a large proportion of skew bridges.

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Fig.2.2. The High Level Bridge, Newcastle on Tyne. A bow-string structure designed by Robert Stephenson.

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Fig 2.3.A cast iron arch railway bridge.

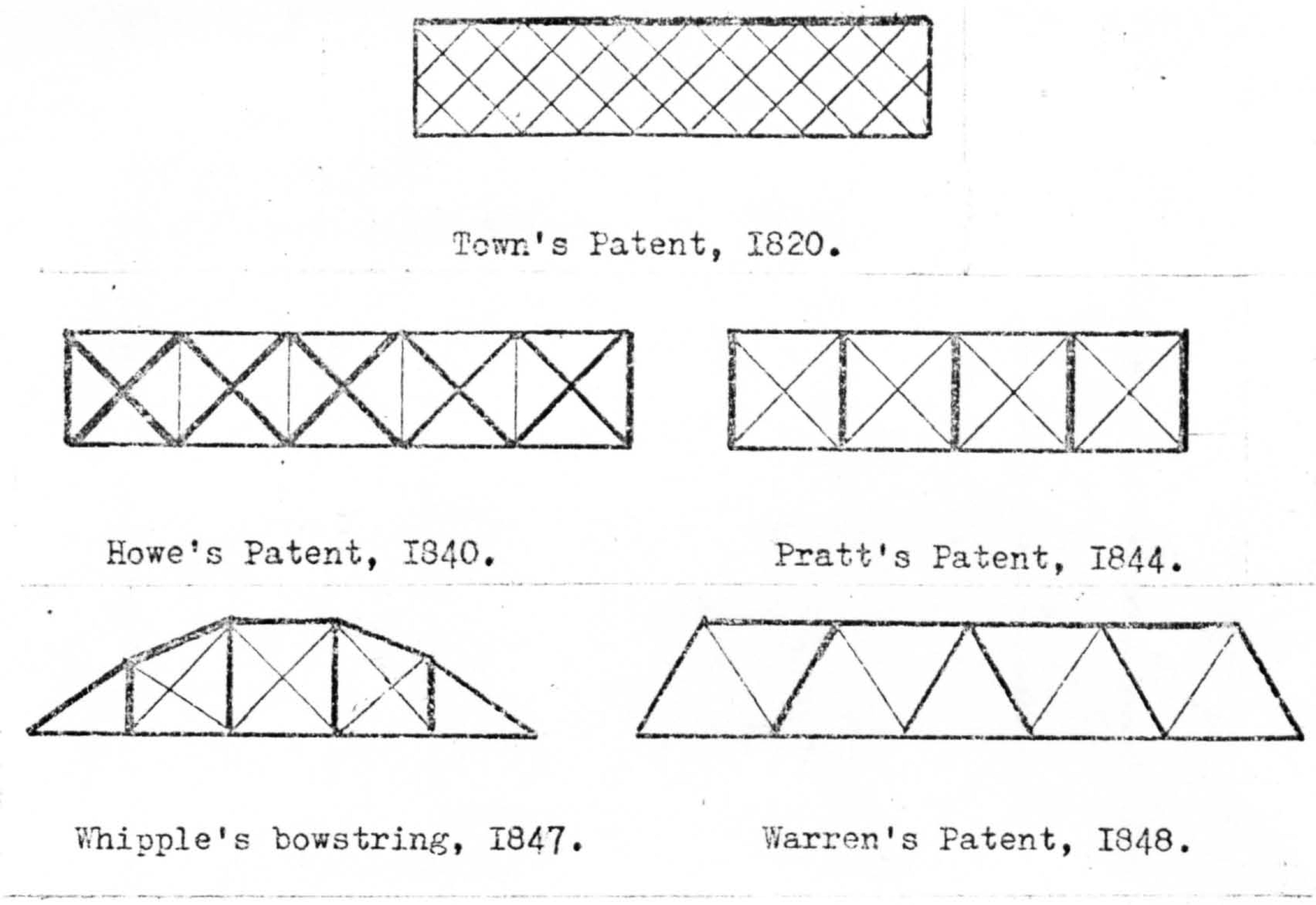

Fig 2.4. Various types of truss bridge.

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Fig 2.5. Early example of a Howe type truss.

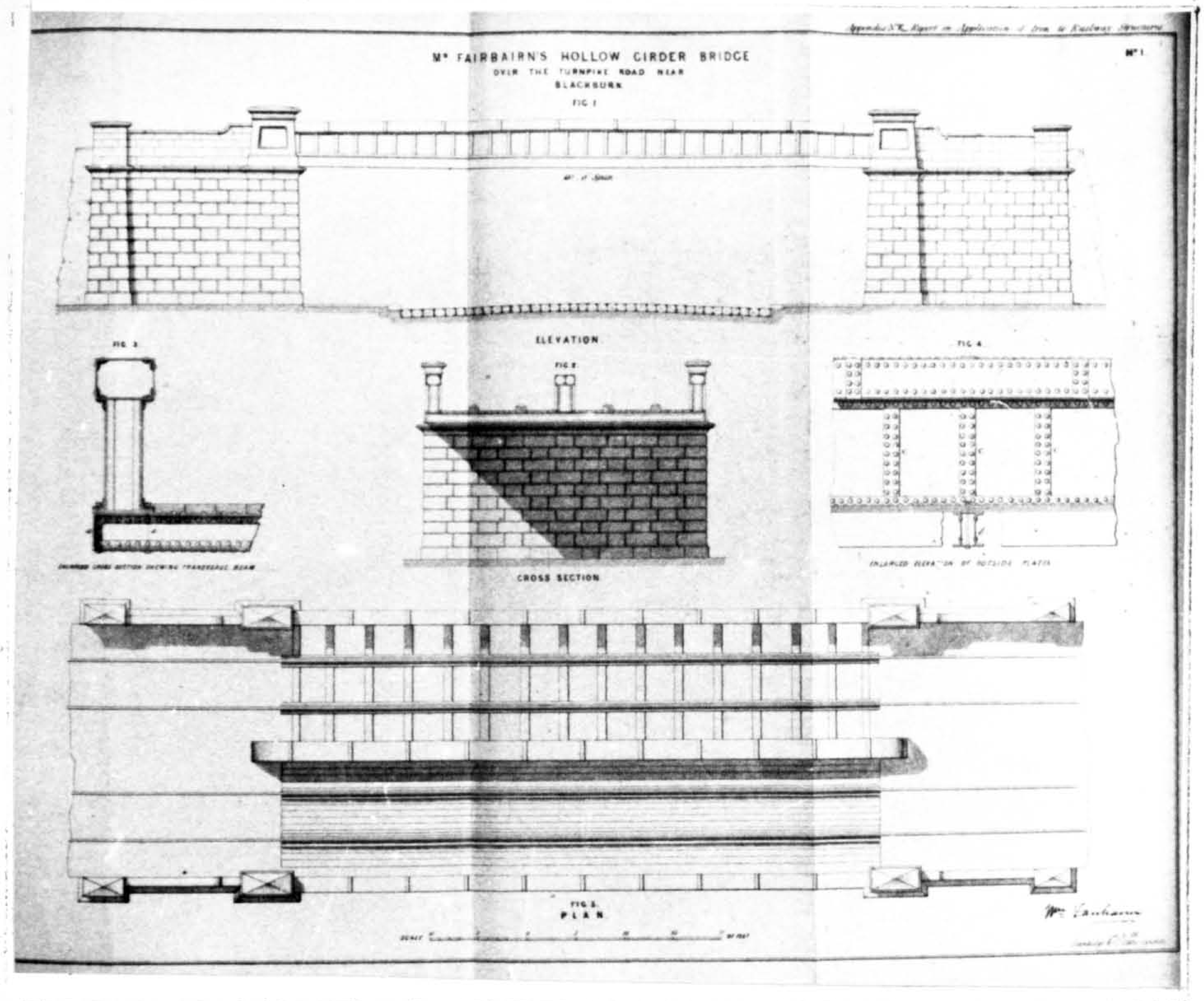


Fig 2.6. Fairbairn's Hollow Girder bridge.

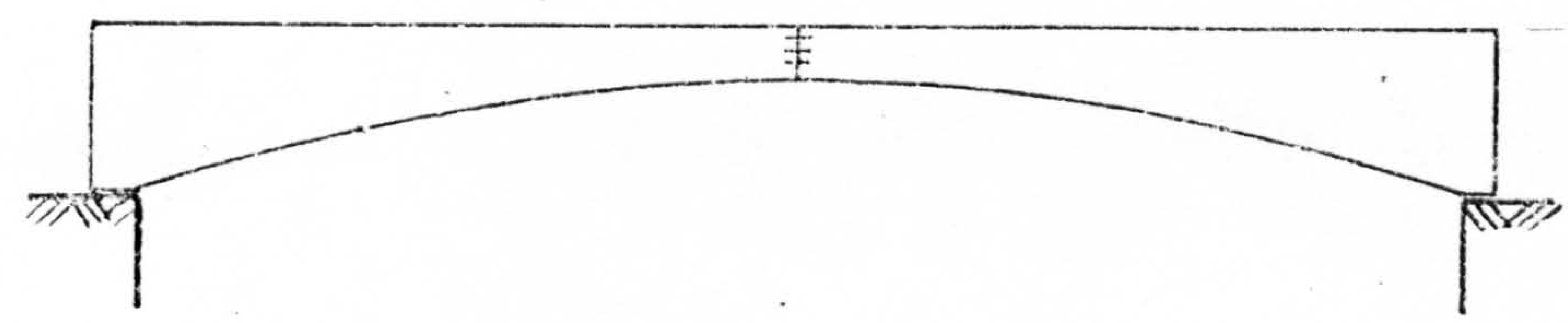


Fig 2.7. a built girder, with bolted central joint.

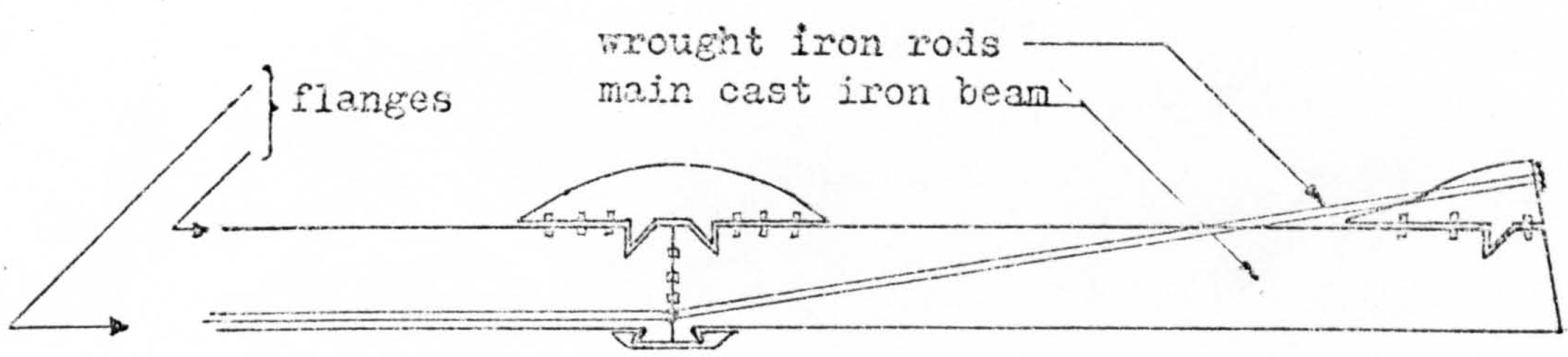


Fig 2.8. Half elevation of a trussed cast iron girder.

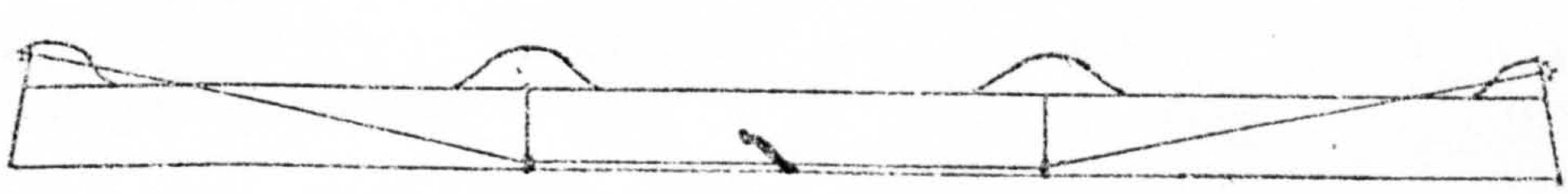


Fig 2.9. Wrought iron rods supporting cast iron girder after failure.

from the earliest days, but the first examples, dating from c.1840 were made of timber and proved insufficiently strong to cope with the imposed loads. Thus they were not widely imitated on this side of the Atlantic. A few lattice and Howe type girders were built on the Dublin-Drogheda railway in 1842, but these were not a success because the joints were inadequate to deal with the loads (fig.2.5). These structures became dangerously springy and expensive to maintain. Robert Stephenson disapproved of them for these reasons and also because in iron examples the top chord tended to buckle - a phenomenon which later caused foreign engineers a lot of trouble.

A few years after the Dee disaster, tubular girders built on the principles outlined in Fairbairn's patent of 1846 provided an excellent way of spanning distances up to 50m. (fig.2.6). Unfortunately when Stephenson was considering the design of the Dee Bridge in 1845, Fairbairn had not completed development of the new form he was evolving from the relatively new technology of riveted wrought iron plates. Thus, in January 1846, when he suggested that tubular girders might be employed for crossing the Dee, the idea was rejected. However, when the firm was first employed, in a 21m (60') span carrying the Blackburn and Bolton Railway over the Leeds and Liverpool Canal (1847), the Fairbairn design showed a 50% saving in materials relative to an alternative trussed cast iron girder.

The built girder (fig.2.7) consisted of separate castings fitted closely at the joints and bolted together. A wrought iron clip was sometimes added over the tension flange to make the system look more secure. By 1845 engineers found that this form was a wasteful way of obtaining long spans, because it was obviously undesirable to stress material used in this way to any very high level.

The trussed girder bridge (fig.2.8) was a development of the built girder, and can be most simply described as sections of cast iron beam bolted together end to end and assisted by a wrought iron trussing system. By 1841 this form of bridge was not uncommon where it was necessary to cross more than 10-13 m (30-40') with a structure of minimum construction depth, typical applications being for railways crossing canals and rivers where economy dictated that the necessary headroom be provided without the extensive approach embankments required for an iron or masonry arch. It was a practical solution to the problem of making long beams from a material with very poor tensile strength and a susceptibility to brittle failure. Whereas for spans of up to 13 m (40') simple cast iron beams were almost invariably used, for longer distances accurate evenly cooled casting was impossible and long distance transport quite impracticable. The first trussed girder structures were built by Charles Vignoles on the Northern Union Railway in Lancashire (see Table 2.2) and were little more than a direct adaptation of what was being incorporated in contemporary iron-framed mills where the function of the wrought iron was only to bind the cast iron in the event of a fracture.[6] (fig.2.9). There is evidence that this idea worked in practice because a span designed by Stephenson in 1839 to cross the River Lea at Tottenham failed after a few months service. G P Bidder (the contractor for the line) reported to the Institution of Civil Engineers in April 1847 that the wrought iron links had saved the structure from collapse; "..The trains ran over the bridge for some time when, without any notice one of the girders broke, where, it afterwards was shown, an imperfection existed in the metal; the tension rods, however, supported the flaw and held the parts so tightly together that the trains passed over as usual until the accident was repaired."[7]

During the period 1831 to 1845 the trussed girder underwent a haphazard development of increasing spans (fig.2.10) culminating in the 30.5 m crossing of the Dee Bridge. Only in the latter stage of the evolution was an attempt made to add to the strength of the girder by prestressing the wrought iron, and then only by Robert Stephenson and his assistants. (asterisked structures in Table 2.2)

TABLE 2.2 TRUSSED CAST IRON GIRDER BRIDGES

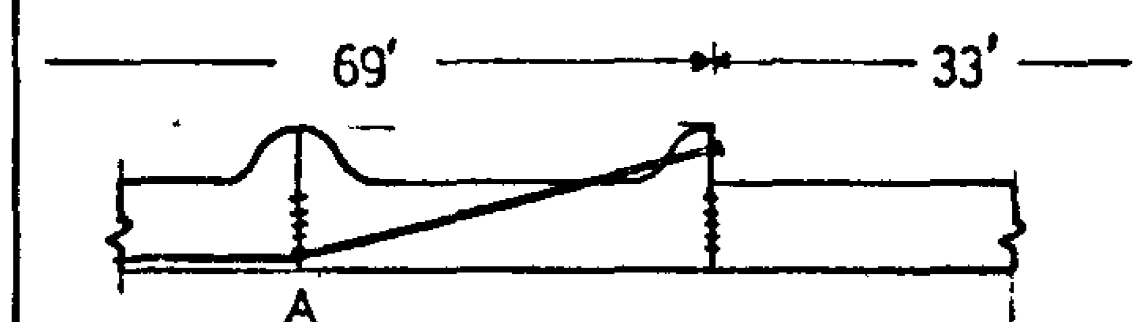
Ref.	Owner) Designer) of Bridge & date Name)	u / o	Span, l, ft. l	l^2	Depth of beams d = in.	Truss System	Bottom Flange w in.	Bottom Flange t in.	Bottom Flange Ab in^2	Top Flange w in.	Top Flange t in.	Top Flange At in^2	(Ab)(d) in^3	Wt	General Notes
															u/o = under or over railway Truss system: either 2 or 3 links, *indicates prestressing. w,t,A = width, thickness & Area Wt = Web thickness Imperial units retained Dates are of the year construction began
	North Union Railway Company des. C B Vignoles 1831														
1.	Wallgate Street, Wigan	u	26	676	30	2	12	2	24	6	2	12	720		tension rods 2" diameter
2.	Chapel Street, Wigan	u	26	676	24	2	8.5	1.87	16	8	1.37	11	384		
3.	Leigh Canal, Bamfurlong	u	32	1020	24	2	8.75	1.75	16	9	1.5	13.5	384		
	London & Birmingham Railway des. R Stephenson 1836														
4.	Lutterworth Bridge	o	64	4100	33		20	1.75	35				1155		
	Eastern Counties Railway des. R Stephenson 1836														
5.	Rye House (over River Lea)	u	69	4760	36	3	18	2	36	8	2	16	1296		Tracing of Drawings in Plan book accompanying Board of Trade 'Returns of Iron Bridges'
6.	Newport ((")	u	55	3025	36	3	18	2	36	8	2	16	1296		
7.	Lea Bridge(")	u	65.25	4356	36	3	18	2	36	8	2	16	1296		 Rye Ho. (sketch). No provision for adjustment of trussing at A. Only 1 truss rod on each side of the girder 3"x1½". Bridges 6 & 7 similar only Lea Br. had truss rods 6"x1"
	London & Blackwall Railway (extension) 1837														
8.	Minories	u	65	4325	36	3	24	2.5	60	8	2	16	2160	2	

TABLE 2.2 (contd.) TRUSSED CAST IRON GIRDER BRIDGES

Ref.	Owner) Designer) of Bridge & date Name)	o / u	Span, l, ft. l	l^2	Depth of beams d = in.	Truss System	Bottom Flange w in.	Bottom Flange t in.	Bottom Flange Ab in^2	Top Flange w in.	Top Flange t in.	Top Flange At in^2	(Ab)(d) in^3	W_t	General Notes
	York & Newcastle Railway 1841														
9.	Durham & Sunderland Road	u	59.5	3540	36	3	24	2	48	8	2	16	1728		) One truss rod each side of girder 6"x1"
10.	" " "	u	59.5	3540	36	3	24	2	48	8	2	16	1728		) Same pattern of castings as Rye Bridge
11.	Whitwell Branch Railway	u	59.5	3540	36	3	24	2	48	8	2	16	1728		)
12.	Simpasture Branch Railway	u	59.5	3540	36	3	24	2	48	8	2	16	1728		)
	Dublin & Drogheda Railway J Mc Neil 1841														
13.	Strand Bridge	u	2x40.5	1640	30		18	3	54	4 dia		12.5	1620		truss rods 2.25 dia. Rods firmly keyed into heavy cast iron standards on abutments & piers
14.	Howth Bridge	u	48	2300	42		18	3	54	6	2.5	15	2268		3.25 dia,
15	London & North Western R'way Macclesfield Branch 1843														
15.	Mill House Bridge, Aldington	u	59.5	3540	54		24	2.5	60	8	3	24	3240	2.5	2 truss rods per girder 6"x1.75" 54" deep at ends. No figure for d_{min}.
	East Lancashire Railway Blackburn & Preston District 1844														
16.	Cherry Tree Bridge	o	37	1370	15		12	2	24	4	2	8	360		
17.	Tram Road Bridge	u	41.5	1720	24		12	1.5	18	8	1.5	12	432		
	York & N.Midland Railway York & Scarborough Branch des. R Stephenson 1844														Drawing at Public Record Office (British Transport Records ref:RAIL 1006/115)
18.	R Ouse, York	u	2x70	4900	39	3	24	2.5	60	8	3	24	2340	2.5	truss rods 6"x1"
	Stockton & Darlington R'way des. R Stephenson 1844														
19.	R Tees, Darlington	u	3x83.25	6930	36	3	24	2	48	8	2	16	1728		truss rods 6"x1" After the Dee Bridge failure, Robert Stephenson recalculated all the trussed cast iron girders designed in his office, & found that this had a lower ultimate:working load ratio than any other.

TABLE 2.2 (contd.) TRUSSED CAST IRON GIRDER BRIDGES

Ref.	Owner) Designer) of Bridge & date Name)	o / u	Span, l, ft.		Depth of beams d = in.	Truss System	Bottom Flange			Top Flange			(Ab)(d) in^3	W_t	General Notes
			l	l^2			w in.	t in.	Ab in^2	w in.	t in.	At in^2			
	Eastern Union, Ipswich & Bury St Edmunds Railway														
20.	Mile End Road 1845	u	30	900	28				6				168	1.5	
	Huddersfield & Manchester R'way														
21.	Rassbottom Street 1845	u	63	3970	40	3	24	2.5	60	8	2	16	2400		2 truss rods per girder. Similar to Rye House Bridge
	Lancashire & Yorkshire R'way Wakefield, Pontefract & Goole division 1845 des. J Hawkshaw														
22.	Calder Bridge	u	3x55.5	3080	46.5		24	2.5	60	8	2	16	2790	2	)
23.	Aggbrig Bar Bridge	u	64.5	4160	46.5		24	2.5	60	8	2	16	2790	2	)
24.	Barnsley Canal Bridge	u	2x59	3480	46.5		24	2.5	60	8	2	16	2790	2	)
25.	Wakefield & Doncaster Rd.Br.	u	64.5	4160	46.5		24	2.5	60	8	2	16	2790	2	) 2 truss rods per girder
26.	Weeland Road Bridge	u	68.5	4690	42		20	2.5	60	11	2.5	27.5	2520	2	)
27.	Knottingley & Goole Canal Br.	u	88.5	7832	54	3	24	2.75	66	8	2	16	3564	2	)
28.	Leeds & Barnsdale Rd.Br.	u	61	3721	46.5		24	2.5	60	8	2	16	2790	2	)
	Manchester South Junction & Altrincham Railway 1845														
29.	London Road Bridge	u	74	5476	42	3	24	2.6	63	7	2.5	17.5	2646	2.5	)
30.	Oxford Road Bridge	u	71	5041	39	"	"	"	"	"	"	"	2457		)
31.	Gloucester Street Bridge	u	67	4489	"	"	"	"	"	"	"	"	"		)
32.	Finch Street Bridge	u	52	2704	"	"	"	"	"	"	"	"	"	2.25	)
33.	Albion Street Bridge	u	52	2704	"	"	"	"	"	"	"	"	"	2.25	)
34.	Knott Mill Bridge	u	66.5	4422	"	"	"	"	"	"	"	"	"	2.5	)
35.	Bridgewater Viaduct	u	66.5	4422	"	"	"	"	"	"	"	"	"		) 2 truss rods per girder
36.	Warehouse Road Bridge	u	66.5	4422	"	"	"	"	"	"	"	"	"		)
37.	Water Street Bridge	u	52	2704	"	"	"	"	"	"	"	"	"		)
38.	Birmingham Street Bridge	u	63	3969	"	"	"	"	"	"	"	"	"		)
39.	Egerton Street Bridge	u	50	2500	"	"	"	"	"	"	"	"	"		)
40.	R.Mersey Bridge	u	66.5	4422	"	"	"	"	"	"	"	"	"		)

TABLE 2.2 (contd.) TRUSSED CAST IRON GIRDER BRIDGES

Ref.	Owner) Designer) of Bridge & date Name)	o/u	Span, l, ft.		Depth of beams d = in.	Truss System	Bottom Flange			Top Flange			(Ab)(d) in^3	W_t	General Notes
			l	l^2			w in.	t in.	Ab in^2	w in.	t in.	At in^2			
	Trent Valley Railway Co 1845														Opening of this railway delayed from June-Dec. 1847 while these bridges were investigated & strengthened
41.	Oxford Canal Bridge	u	43.5	1892	30		24	2	48				1440		2 tension bars per girder 6"x1"
42.	Coventry Canal Bridge	u	60	3600	36		"	"	"				1728		" "
43.	R.Tame Bridge	u	3x70	4900	36		"	"	"				"		" "
44.	T.N.R. Handsacre Bridge	u	57.2	3271	36		"	"	"				"		" "
45.	Canal at Handsacre	u	54	2916	30		"	"	"				1440		" "
46.	Canal at Colwick	u	55	3025	30		"	"	"				"		" "
	York & N.Midland Railway Church Fenton-Harrogate Branch des. R Stephenson 1845														
47.	R.Wharfe Bridge	u	2x69.25	4795	39	3	24	2.5	60	8	3	24	2340	2.5	tension rods 6x1.75) similar design to
48.	Wetherby turnpike road	u	51	2601	33	3	18	3	54	4	3	12	1782	2.5	) Rye House Bridge
49.	York & Bridlington turnpike rd.	u	56	3136	36	3	21	2.5	52	8	3	24	1872	2.5	tension rods 6x1)
	Shrewsbury & Birmingham R'way 1846														
50.	Hadley Road Bridge	u	50	2500	36										
	Chester & Holyhead Railway des. R Stephenson 1846														
51.	R.Dee Bridge	u	98	9604	45	3	24	2.75	66	7	2	14	2970		tension rods 6x0.31, 8 on each side of the girder
	Midland Railway des. W H Barlow 1847														
52.	Canal Bridge at Lenton	u	25.75	663	18	2	7	2	14	7	1.25	8.7	252		Cast iron I beams with strong cross/bracing
53.	Cranfleet Cut Bridge	u	31	961	18	2	8	1.75	14	8	1.75	14	252		tension rods 1.75" diameter
54.	Leicester, Jail Lane Bridge	u	30.5	930	17	2	8	1.75	14	8	1.75	14	238		

TABLE 2.2 (contd.) **TRUSSED CAST IRON GIRDER BRIDGES**

Ref.	Owner) Designer) of Bridge & date Name)	o / u	Span, l, ft. l	l^2	Depth of beams d = in.	Truss System	Bottom Flange w in.	Bottom Flange t in.	Bottom Flange Ab in^2	Top Flange w in.	Top Flange t in.	Top Flange At in^2	(Ab)(d) in^3	Wt	General Notes
	Manchester, Sheffield & Lincolnshire Railway des. J Fowler 1847														
55.	Old Ancholme Bridge	u	69	4761	60	2	30	2.75	82.5	4	2	8	4950		tension rods 9"x2.5"
	Proposed in 1847 :-														
	Leopold Railway (Italy) des. R Stephenson														
56.	R.Ombrone Bridge	u	96	9216	96	3	24	2	48	24	2	48	4608		fig. 2.24
57.	R.Arno Bridge	u	96	9216	84	-	18	1.62	29	18	1.62	29	2436	1.75	fig. 2.25

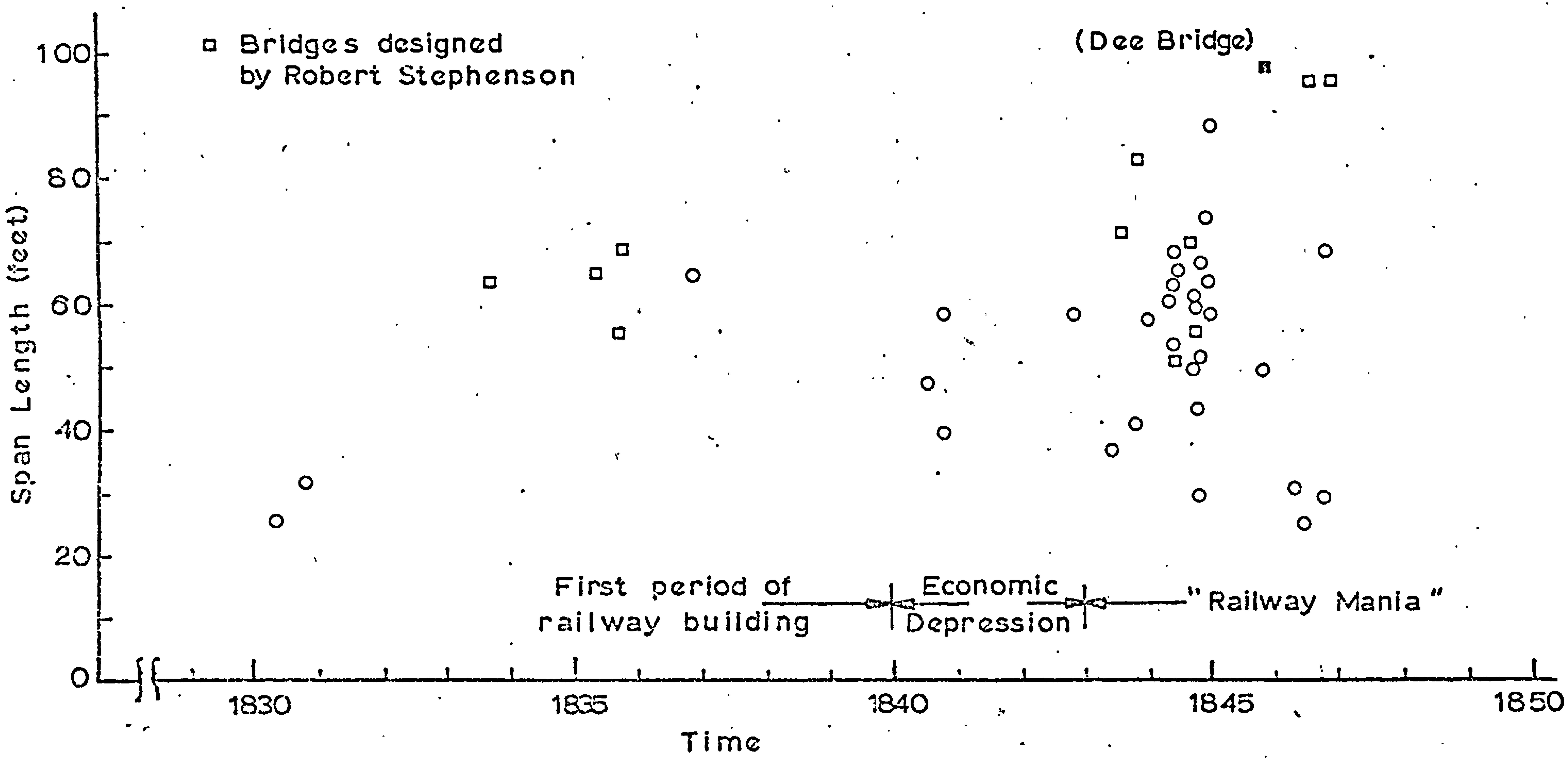


Fig 2.10. Graph showing the distribution of trussed girders with time.

2.2 History of the Design

When the Dee Bridge was designed in 1845, Stephenson had to choose between the various types of structure just described. In fact the range was rather narrower than suggested, because the embryonic state of the tubular and truss girder forms ruled these two out as serious possibilities. Also, by the date in question the built girder and cast iron arch had more efficient counterparts (for medium spans) in the trussed cast iron and bow and string girders respectively.

On the whole it is surprising that Stephenson did not use bow and string girders, because, with the Chester and Holyhead directors ready to pay huge sums for crossing the River Conway and the Menai Straits he could have been expected to aim at providing something outwardly more solid and durable at the Dee Crossing that a trussed girder.

Why then, did he use this type of structure? Probably because there was a fashion for trussed girders at the time of the railway mania (1844-50) presumably derived from the fact that these bridges symbolised something of the subtlety of modern technology.

They had not been popular during the first period of railway building (1825-1840) as can be seen from Table 2.1. which indicates the distribution of underline iron bridges on the railway network in 1847. Confirmation of this view is found in the only major book on bridges of the period, Weale's treatise of 1843, in which the forms recommended for underline structures are masonry and cast iron arches and timber trusses. There is no mention at all of trussed cast iron girders.

Stephenson (or rather his assistant, Charles Wild, who probably carried out the design under minimal supervision) was probably swayed by three considerations in making his final choice. Firstly a trussed girder bridge could be built economically and quickly, thus opening the line for the carriage of building materials. Secondly, fear of pier subsidence demanded light, simply-supported spans. Lastly, the trussed girder used in 3, 30.5 m (98') spans offered the engineer the opportunity to achieve every bridge-builder's secret desire, which is to build the longest span constructed in a particular form.

Figures 2.11 to 2.13 show the important features of the bridge as constructed in 1845-1846: they are mostly taken from the report prepared by Mr Walker and Capt. Simmons after the accident.

The 30.5 m (98') spans were achieved using large cast iron components up to 11 m (36') long. These were very nearly the largest size that could be satisfactorily manufactured, and in the completed structure some of the castings had visible distortion, probably due to the mould being inaccurate or to the metal cooling unevenly. The need to pour the metal into the mould quickly in order to prevent this latter effect meant that it was difficult to exclude pockets of air from the castings, so it was hardly surprising that after the collapse a representative of the Mausley Iron Company, who made the ironwork reported that a casting flaw had been discovered in one of the girders ("mere honey-cake" he called it).[9] Such was the frequency with which these blowholes occurred that he had thought nothing of it and followed the generally accepted practice of filling the cavity with a mixture of iron filings and wax, assuming that the safety margin was sufficient to cover the loss of strength.

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Fig 2.II. Elevation of the Dee bridge (part).

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Fig 2.I2. Elevation of Dee bridge girder.

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Fig 2.I3. Cross-section, Dee bridge.

The main members of the bridge were calculated as simply supported cast iron girders proportioned in accordance with Hodgkinson's formula for the design of cast iron beams.[10] This expression, which will be discussed more fully later, was deduced from experimental results and was supposed to apply only to conventional I beams and static loading. No research had been done to investigate the effect of trussing the beam with wrought iron straps nor to assess the effect of varying the beam section, so the application of the design rule to this type of structure was fundamentally invalid. Nevertheless it was not considered necessary to assess the beam and truss interaction (fig.2.14) quantitatively when the design was made. Indeed before the stiffness method of structural analysis (pioneered by Navier in his "Résumé des leçons" in 1826) caught on in the late 1800s such work was difficult. As matters stood, the designer assumed that there would be a certain amount of beneficial composite action between the two systems and that the section of the cast iron could be reduced from that nominally required for carrying the whole load by bending action. The extent of the reduction was to make the strength of the cast iron beam only 1.55 times the design load instead of the more usual figure of 4 or 5.

The beams had an inherent weakness in that they were made of several pieces bolted together, another move not envisaged by Hodgkinson. The bolt holes, especially those through the flanges must have reduced the strength of the beam and it is hard to believe that a satisfactory bearing was obtained between all the various pieces, despite the introduction of layers of felt along the joint lines.[11] Furthermore, holes were cast in the compression flange for the positioning of ornamental eagles at the quarter points of each span.[12] Quite apart from weakening the flange in themselves, these were left empty, so that it

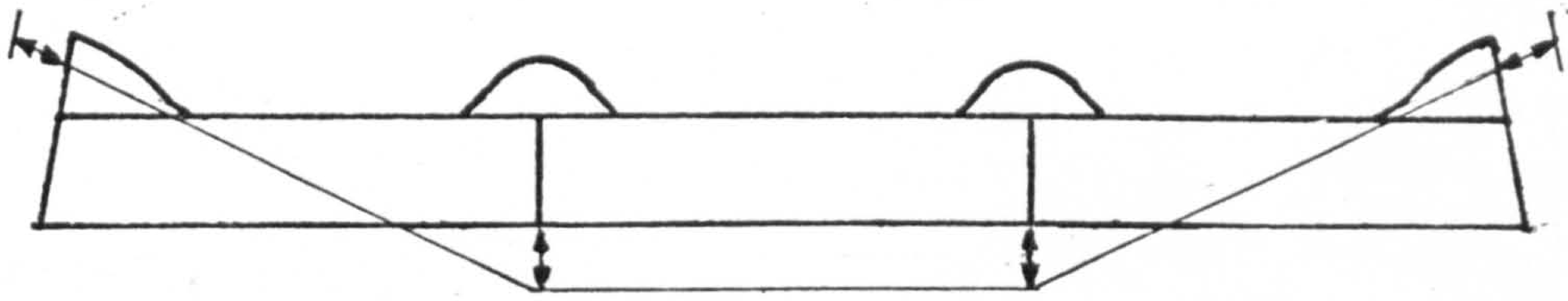

Fig 2.I4. Interaction of beam and trussing.

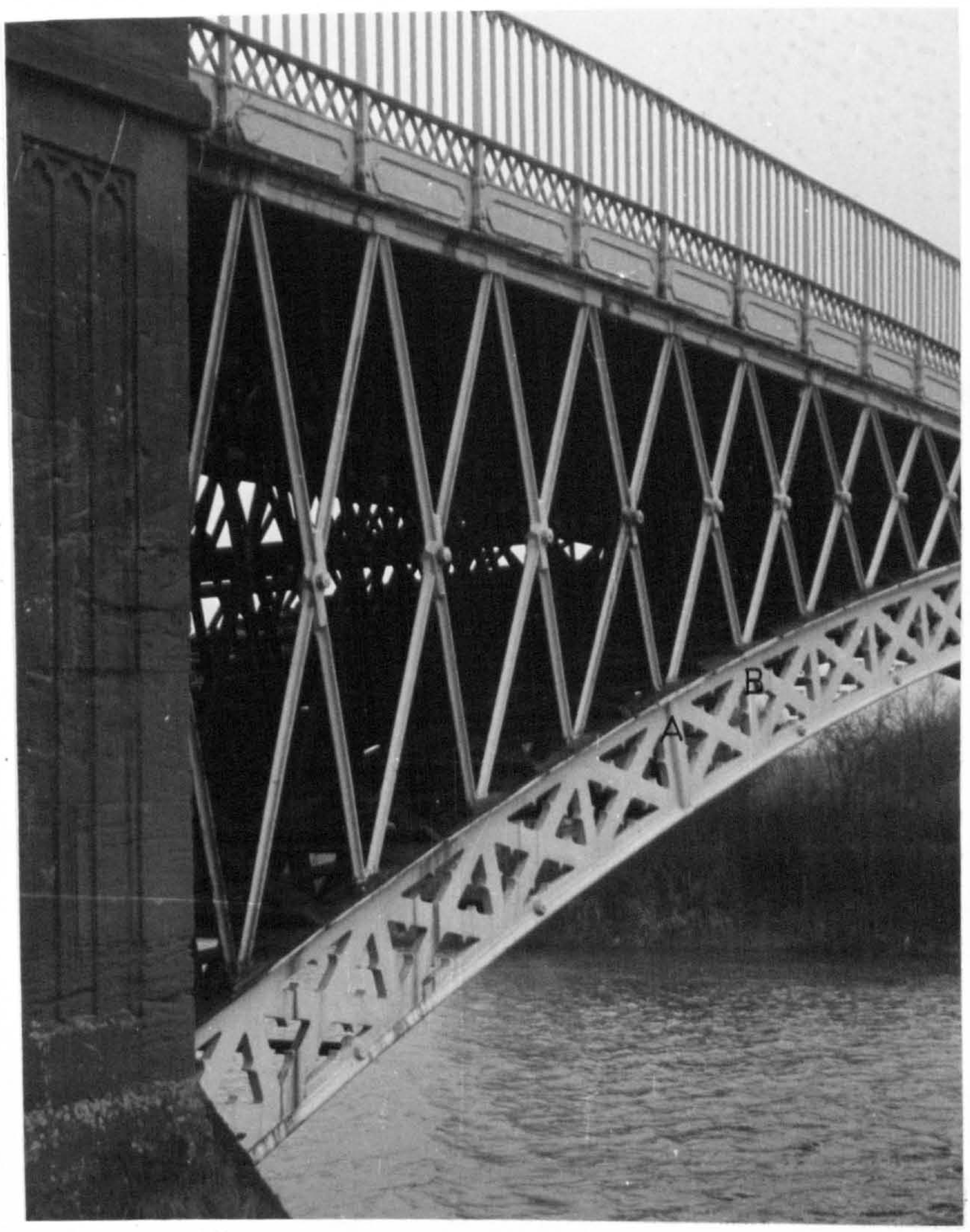

Fig 2.I5. Frost damage, Mythe bridge.

At A. and B. steel plates have been bolted onto the main arch ribs following local fracture due to water freezing in the sockets cast to receive the spandrel struts.

is quite likely that water froze in them during the only winter the bridge survived, causing local damage to the cast iron. The author has noticed similar damage on other early cast iron structures, notably Telford's Mythe Bridge at Tewkesbury (fig.2.15). Mention should also be made of the facts that the bridge abutments were built parallel to the flow of the river and that the railway was on a slight curve. The first of these matters has always been common enough practice although it means that one girder is loaded before the other. The curve of the rails added to the uneven distribution of loading on the girders, introducing a centrifugal force of about 3.3 kN (750lbsf) for a normal train as well as an uneven static distribution of load.[13] A common way of counteracting these effects was by bracing the main girders firmly together, but the binding system applied to the Dee Bridge was flimsy and ineffective, consisting only of wrought iron ties dovetailed into sockets cast on the main girders.

The train weight was modelled as a uniformly distributed load of 33kN/m (1 ton/ft). This was usual at the time, but it was a dangerous practice because it did not recognise that railway loads were significantly different from horse drawn vehicles and herds of animals allowed for in the previous generations of structures. Unlike the old cast iron arches and aqueducts, the new bridges had to carry fast moving heavy locomotives along a track which in the early days was so primitive and so irregular that considerable impact forces were imparted to the structure.

The transmission of these forces to the bridge girders was also unsatisfactory (fig.2.13). The cross-bearers that supported the bridge deck and the track ended in cast iron shoes resting on the bottom flange of the main girders. This resulted in a highly undesirable condition of shear in the flange, and also in the development of torsion in the deck, sufficient in practice to cause a 75 mm (3") deflection of the girders inwards as a train passed overhead.[14]

This outline of the Dee Bridge would not be complete if attention was not drawn to the fact that the structure was unusually daring, not only because its span was longer than any earlier examples but because the other geometric parameters were devised so that both cast and wrought iron were used exceedingly sparingly. From Table 2.2 and fig.2.16 it can be seen that the girders were unusually shallow and that the section of the top flange was not increased to sustain the additional compression thrown upon it by the prestressing.

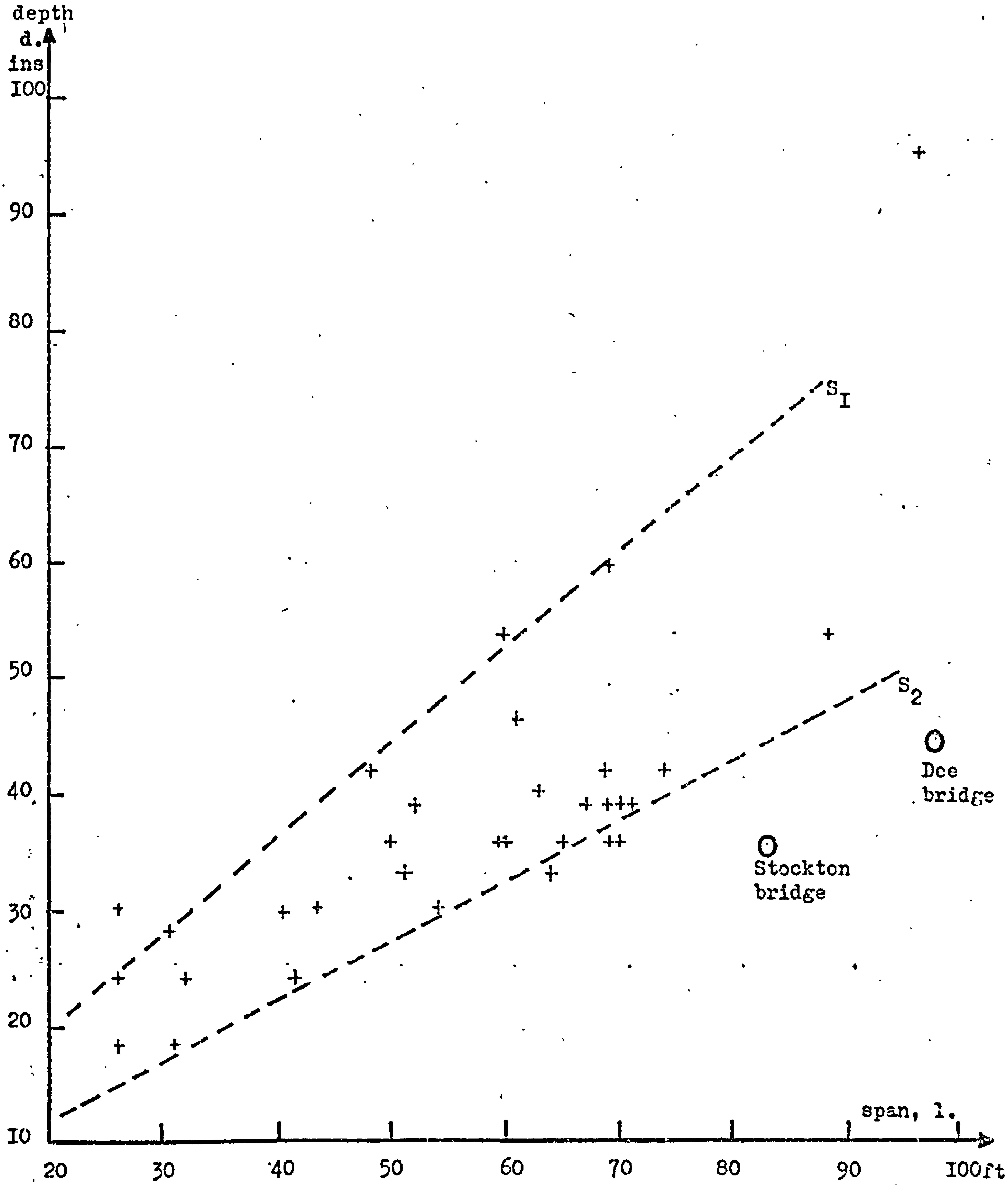

Fig 2.I6. Graph of depth of girder v. span, showing normal limits of practice (S_I and S_2).

2.3 The Collapse

The ironwork of the bridge was erected in the summer of 1846 and one line of rails was ready for service in September of that year. This enabled the contractors to carry materials to sites further along the line to Holyhead, and it is recorded that one of the earliest trains to cross the structure consisted of heavily-laden wagons drawn by three thirty ton locomotives. Early in October one girder casting was found to be cracked at one of the third-point joints, but the fractured metal was held together by the wrought iron trussing until the opening was propped and a replacement girder substituted.[15]

The passenger train service from Shrewsbury to Chester was inaugurated on October 31st and ran satisfactorily for six months. The Bridge behaved satisfactorily, its only noticeable deficiency being that it was very prone to vibration as trains crossed overhead. In a test carried out on the intact line of rails after the collapse, Mr Walker and Capt. Simmons noticed that "there is a shaking and oscillating motion caused by an engine going at considerable speed, the effect of which depends on degree and is difficult of calculation, but is a constantly repeated force which upon a hard rigid body like a cast iron beam tends to weaken and injure it. In addition to this is the tremulous motion of the beam when the engine is going over. This tremor was so sensible that Capt. Simmons could not distinctly see the edge of the beam."[16]

In May 1847 it was decided that the bridge superstructure should be somewhat modified to protect its timber decking from the risk of fire started by cinders falling from the fireboxes of passing locomotives. During the afternoon of May 24th a permanent way gang spread a 130 mm (5") layer of sandstone ballast over the decking thus increasing the weight supported by each pair of girders by about 25 tons. The work was personally inspected by Stephenson, who happened to be in Chester on business.

The job was finished in time for the 6.20 pm train from Chester to Shrewsbury. The locomotive and its five flimsy carriages were travelling at about 11 m/s (25mph) as the train reached the curved line of rails across the bridge. The first two openings were safely cleared but on passing over the third span from Chester the engine driver felt the rails giving beneath him. To quote from his report:-[17]

"We passed over two arches of the Dee Bridge without noticing anything unusual, but on reaching the third I felt a 'sinking'. Fearing something had given way I gave the engine all the steam I could, which caused here to jump up and break away from the tender. I then saw that Anderson (the stoker) was on the tender. He was standing on the offside, but in a moment or two he was thrown off the engine and his head fell on the rails at the spot which the jury has viewed. As soon as I could stop the engine I looked back and wondered what had become of the train, but, supposing the bridge had fallen in, I proceeded to Salteney Junction, where I beckoned some plate layers who were leaving work. I could not tell them what had taken place for I did not exactly know."

Alex MacGregor, who finished up in the river surrounded by the remains of what had been the luggage van near the rear of the train was sufficiently unruffled by the collapse to remember that the train did not seem to leave the rails before the sinking began. On crossing the bridge he heard a crash and noticed an instant deflection of the carriages.

The accident was seen by a little boy who was fishing the river from the Wrexham bank near the bridge.[18] Hearing a train coming he naturally looked up, and on its crossing the span nearest to him he heard a crashing noise for two or three seconds and then saw the four carriages come down "in a string". (fig.2.17)

Despite the fall of 11 m (36') into the river, of the 25 people in the train only five were killed, miraculously few fatalities for such a severe accident. A further 18 were injured. A Coroner's Inquest was held to establish the manner in which the dead were killed, but because this was the first serious structural collapse on the railway network, the Court took on the aspect of a trial of engineering expertise by a hostile public. As a result its proceedings attracted widespread interest. The Illustrated London News, The Civil Engineer and Architect's Journal, Railway Chronicle, Railway Times, Railway Record, Herapath's Journal and Bradshaw's Railway Gazette all reported the hearing in considerable detail, and many prominent engineers attended in person.

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Fig 2.17. The wreck of the bridge.

2.4 The Inquest

A great deal of phoney evidence was given at the Inquest by the various parties trying to avoid liability for the accident. The servants of the Chester & Holyhead Railway Company were instructed to propound the story that destruction of the bridge was directly attributable to a derailment, although the eye witness accounts already quoted negate this possibility. The Chester & Holyhead statements are of limited value because after the verdict on the fatalities had been delivered at Chester, the Company's solicitor, Mr Walker, confided to a friend that only legal expediency had prevented an admission that inadequate knowledge of structural behaviour was responsible for the underdesign of the bridge.[19] It is nevertheless interesting to include a brief sketch of the Chester & Holyhead evidence to show how a deliberate attempt was made to twist the facts and avoid responsibility.

Stephenson of course had to take a leading part in this. He inspected the wreckage on Thursday 27th May and immediately drew up a report for the Chester & Holyhead Directors.[20] After affirming his faith in the trussed girder principle he described the damage and came to the conclusion that fracture I in fig. 2.18 "..was precisely such as might be expected from a large piece of vertical rib being broken out of the girder and separated from the lower flange which remains perfect; a description of fracture which could scarcely be produced by a vertical action ..". He then clumsily manipulated evidence provided by the collapse to show that a carriage wheel had broken and caused a derailment resulting in either the tender or a coach hitting the girder at I with sufficient momentum to break out the shaded chunk of metal.

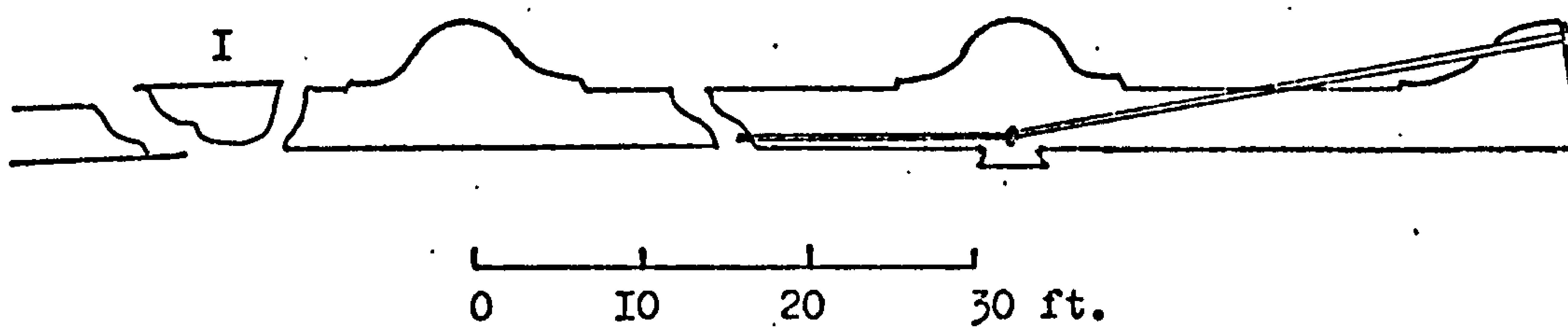

Fig 2.I8. fractures of the broken Dee bridge girder.

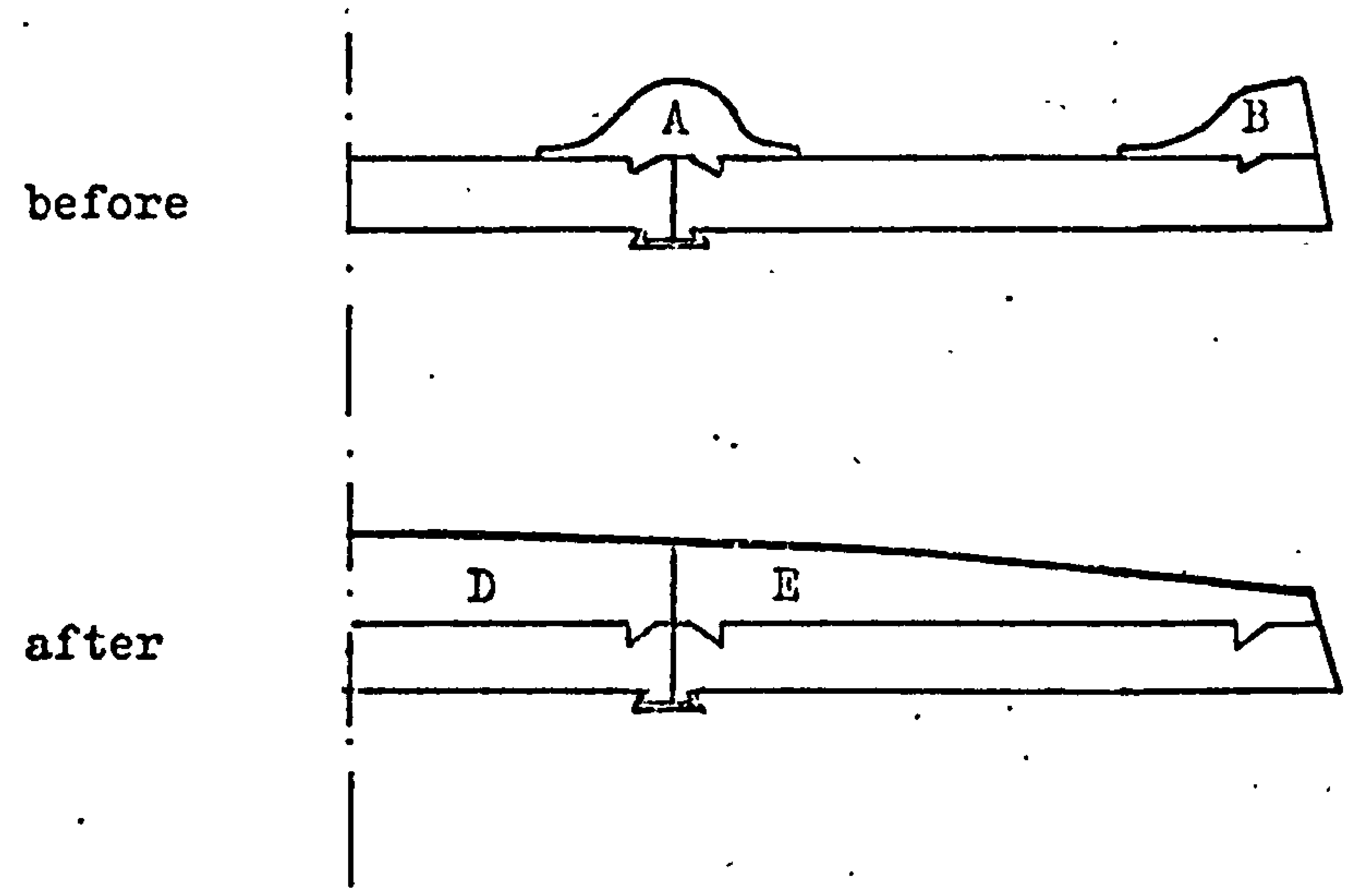

Fig 2.I9. sketch of the modifying castings fitted to the Dee bridge girders.

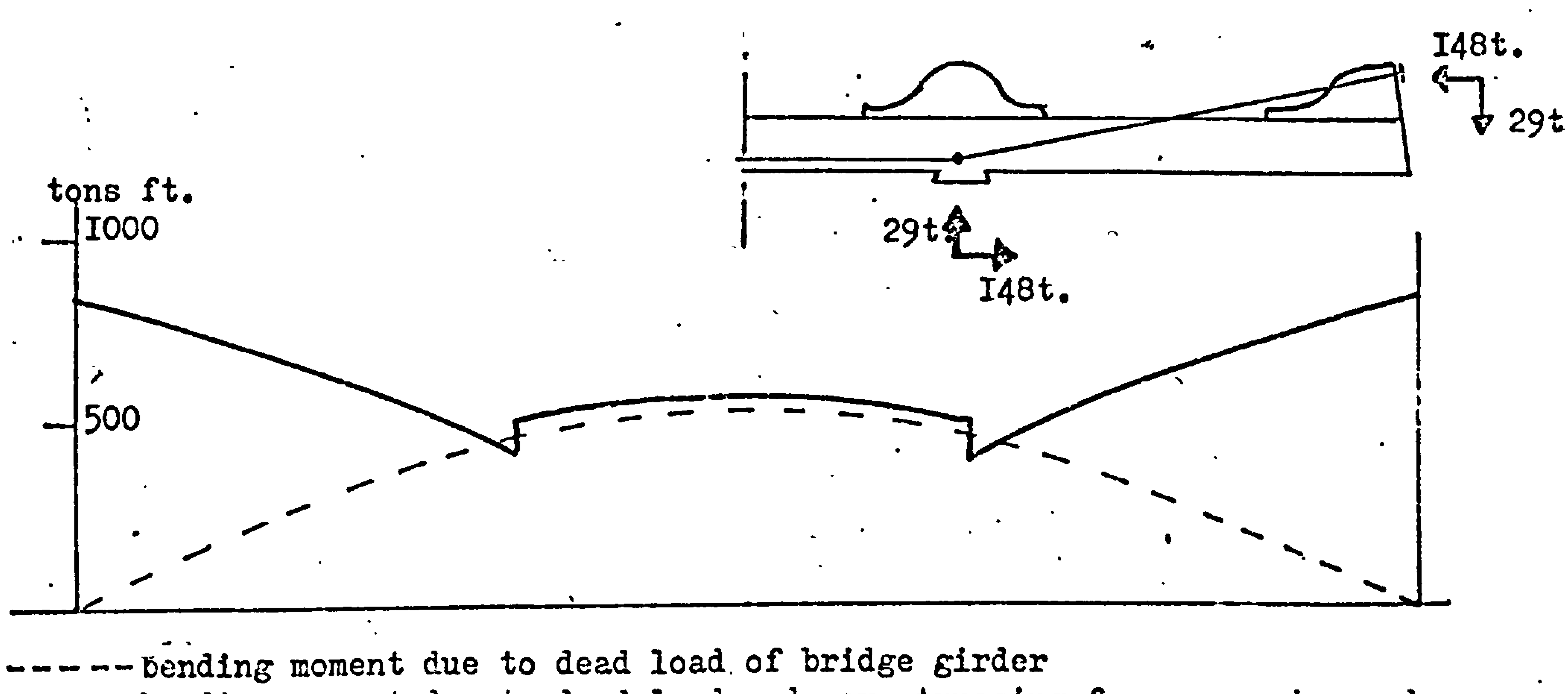

Fig 2.20. Trussing forces and loading distribution, Dee bridge girder.

This view was confirmed by the Clerk of Works on the line when he gave verbal evidence at the Inquest. On June 3rd, Stephenson also had to speak from the witness box, and although the wreckage of the bridge was still in the river,[21] he offered new and amazingly detailed "evidence" to support his theory.

He was backed up by his eminent friend Joseph Locke, engineer of the Grand Junction Railway. Locke expanded Stephenson's theory past all reasonable credulity as he described the course he considered the tender to have taken across the doomed span.[22]

Stephenson also produced a model to illustrate the trussed girder principle. This made the bridge look much sounder than it actually was, and his explanation seems to have been deliberately calculated to mislead the amateur jury.[23] "..It has been said that the links did not aid in sustaining the rigid fabric. That opinion was submitted to me twelve months ago. I have considered the objection and caused experiments to be made. The result is conclusive that wrought iron links are both effective and act in concert. Even so the structure has been varied to meet this objection." He then explained the principle from a model held in his hand. "The weight of the girder causes a deflection of two inches (50mm), but the suspenders can be screwed up to bend it two inches upward. In this state the whole weight passing over the bridge is placed on the suspenders and the wrought iron really takes the place of the cast iron girder. This is what I mostly rely on ... Nevertheless the pair of girders are capable of carrying 280 tons (2.8.MN) of dead weight spread over the whole of the platform. The greatest weight that can possibly be run over the bridge by railway traffic is 170 tons (1.7 MN), but this is an extreme case. There are clearly 100 tons (1.0 MN) to spare, treating the bridge as a pure girder bridge without suspenders ...The wrought iron links are capable of sustaining

129 tons (1.29 MN) uniformly distributed over the bridge, and double that for the whole system of castings and ironwork for one railway. The two principles can not be brought into strict union at one and the same time but they may mutually aid each other ... I have erected in 20 years more iron bridges than any other member of the Profession. ... and I have always acted on Hodgkinson's experiments."

This is confusing for listener and reader alike, as indeed it was intended to be. Fortunately the members of the Chester Jury were not misled by this nor the tales of deficient masonry, sudden subsidence and unequal coefficients of expansion of cast and wrought iron. They took more notice of Messrs. Walker and Simmons, the two engineers sent by the Commissioners of Railways to investigate the accident. These experts said that they were not convinced by Stephenson's demonstration and stated that in their opinion it was very difficult to ensure composite action betwen the cast and wrought iron girders.[24] They said that the cast iron should have been strong enough in itself to carry the whole load with a minimum safety factor of three in place of the value adopted (1.55).[25] Their view was that the cause of the collapse was fatigue of the metal. To use their own words, "When a weight, partly permanent and partly passing but together forming a very considerable proportion of the breaking weight of the girder is in continued operation, that girders of cast iron suffer injury, and their strength becomes reduced."[26]

Consequently when the pompous foreman of the Jury, Sir Ernest Walker was called on by the Coroner to return a verdict it was obvious that the fundamental message had been driven well home:

"We find," he said, "that George Roberts, John Matthews and Charles Nesbitt were accidentally killed on the evening of the 24th of May last, in the Parish of St Mary on the Hill, in the City of Chester, by being precipitated along with a train of carriages on the bed or bank of the River Dee, from the breakage of one of the 12 cast iron girders constituting the Railway Bridge over that River. We find also that Isaac Powis died on the 26th of May aforesaid from injuries he received at the same time and place and from the like cause; and we find that Thomas Anderson came by his death on the 24th of May aforesaid, by being accidentally thrown from the tender onto the rails. We are further unanimously of the opinion that the girder did not break from any lateral blow of the engine, tender, carriage or van, or from any fault or defect in the masonry of the piers or abutments; but from its being made of a strength insufficient to bear the pressure of quick trains passing over it. We feel that the eleven remaining girders, having been cast from the same pattern and of the same strength, are equally weak, and consequently equally dangerous for quick or passenger trains as was the broken one. We consider we should not be doing our duty towards the public if we separated without expressing our unanimous opinion that no girder bridge of so brittle and treacherous a metal as cast iron alone, even though trussed with wrought iron rods, is safe for quick or passenger trains; and we have it in evidence before us, that there are upward of 100 bridges similar in principle or form to the late one over the River Dee, either in use or in the course of being constructed on various lines of railways. We consider all of these unsafe, more or less in proportion to the span; still, all unsafe. We therefore call upon Her Majesty's Government to institute such an enquiry into the merits or demerits of these bridges, as shall either condemn the principle or establish their safety to such a degree that passengers may rest fully satisfied there is no danger ..."

The Coroner said that only the portion of the verdict which related to the death of the victims was acceptable as a verdict. He did however undertake to forward their other recommendations to the Railway

Department of the Government. This resulted in the survey of iron railway bridges previously alluded to. The Board of Trade sent a circular to each Railway Company asking for full details of all iron structures, and as the replies were received it became apparent that there were many anomalies in contemporary practice.[27] These, together with Mr Walker's and Capt. Simmons' report on the Dee Bridge accident led to the setting up of a "Royal Commission appointed to Inquire into the Application of Iron to Railway Structures" (Aug. 27, 1847). The Commissioners' report published in 1849 revealed much about the state of engineering practice and theory and added considerably to them, the most notable contribution being Hodgkinson's investigation of the fatigue characteristics of cast and wrought iron bars and beams. The Report also revealed a need for properly organised engineering education and research, and it thereby stimulated the setting up of University and other schools of engineering.

To round off this picture of the events directly related to the Dee Bridge collapse, it is worth noting the subsequent history of the trussed girder.

After the failure, public opinion demanded that both the rebuilt structure and other bridges of the same type should be strengthened. Some engineers realised that the original idea of the trussed girders was unsound and favoured rebuilding rather than tacking on strengthening material..

In fact a compromise solution was applied between these extremes, i.e. the bridges were visibly strengthened to allay immediate public anxiety, but were replaced at the earliest possible opportunity. A diary of the life of the Dee Bridge is broadly typical of the history of many of these bridges after 1847:-

25.5.1847	Failure of the Dee Bridge
24.6.1847	Undamaged spans supported on timber centering
July 1847	Capt. Simmons inspected the structure and allowed it to reopen, subject to a speed restriction.
11.8.1847	Cost of the temporary support given as £1,000.
Sept.1847	£12,727 paid by the Chester & Holyhead Railway in compensation for injury and death.
6.10.1847	Stephenson reported a test on one of the undamaged girders, which failed at a lower load than anticipated.[28]
23.3.1848	As a witness before the Royal Commission, Stephenson explained how he had strengthened the Dee Bridge (fig.2.19). The unsightly castings A & B were removed and replaced by pieces D & E.[29]
1848	After the collapse, no new trussed cast-iron girders were designed by Stephenson or anyone else.
1853	Stephenson, writing in the Encyclopaedia Britannica, explained why he abandoned the trussed girder.[30] "The objection to this girder is common to all girders in which two independent systems are attempted to be blended; and as a general principle all such arrangements should be avoided. It is useless to say more on the subject of this form of girder as, since the adoption of wrought iron for girders they have been entirely superceded. They were designed when no other means existed of obtaining iron girders of great span, and the melancholy accident which occurred at Chester is the only existing instance of their failure."
1870-71	The Dee Bridge was replaced by wrought iron trusses, supplied and erected by Woodhall's of Dudley.

2.5. The Real Cause of the Accident

Having established a valid cause for the accident, the subject might be considered closed were it not for some further evidence, overlooked at the Inquest, which suggests that there was also something very unusual about the actual mode of collapse, which lay quite outside the experience of all but a few of the contemporary designers.

The line taken by Walker and Simmons was that repeated cycles of loading led to a fatigue failure by some unspecified mechanism but probably by a straightforward compression or tension failure of one of the flanges. This view was corroborated by other engineers present at Chester.

However, a modern observer would say that the beam would probably buckle out of its vertical plane at a lower load than that required for failure by one or other of these modes. Since this would give further weight to the idea that the collapse fits the pattern of development whereby a design principle is extrapolated from its original boundary conditions for use in applications of such size that other failure criteria apply, the notion merits further investigation.

Firstly the modern hypothesis must be tested by reference to what the contemporary observers saw but failed to understand. It is found that at the Chester Inquest there were hints that one or two people appreciated that a buckling mechanism might have been responsible for the failure. Mr Kennedy, partner in the Liverpool firm of ironmasters Bury and Kennedy said that "In experimenting upon beams, I have found that when the sectional area of the upper rib was too small to stand the compression put upon it, similar fractures have been made as the triangular one in this girder, which have been sprang out"[31] (see fig. 2.18)

Henry Robertson, engineer to the Shrewsbury & Chester Railway Company, reinforced this view by suggesting that the compression in the upper flange was actually increased by the wrought iron rods, thus increasing the susceptibility to buckling; "The fracture ... I consider to have resulted from the weakness of the top flange which was compressed and broken by the strain arising from the rolling weight of the engine and tender, and the vibratory motion of the structure itself ... This compression is remarkably evident by the bulging out of the metal at the point of the parting at the top of the web ... In estimating the strength of the girder, I am of the opinion that the tension rods, from the form of the section of the girder, weakened it, and threw an undue strain by compression on the top flange ..."

Stephenson probably grasped the significance of what was said, although the Jury missed it. As a result he had one of the remaining girders tested after the accident. This failed at a static load of 38 tons, far below the 140 tons the designer envisaged as the minimum it should carry.[32] This was proof enough that there was something unusual about the action of the beams, especially as the broken pieces were found to be free from casting flaws.

Suddenly Stephenson comprehended the truth, which was that as Robertson had suggested, the wrought iron links did work against the beam action of the cast iron. When he was called before the Royal

Commission in 1848 he explained that he could now see that, "The fulcrum was below the line of thrust, and increased the tension of the bottom and correspondingly increased the compression of the top by adding to it ..."[33], but even at this stage he does not seem to have realised that this would lead to a buckling mechanism rather than the simple compression or tension fracture predicted by Hodgkinson.

For the sake of clarifying the issue, the author reanalysed the girder along modern lines to see if the breaking load for a buckling mechanism would be lower than that required for a simple compression or tension failure. Fig. 2.20 shows the possible adjustments of the beam truss system and indicates the forces induced by tensioning. In a situation such as this with a small angle of inclination for the wrought iron rods, quite considerable horizontal forces can be induced by relatively little prestressing. This not only brings an undesirable addition to the bending moments in the end portions of the beam, but it also increases the compression forces in the top flange, as suggested by Robertson. The loading distribution that probably existed in the two systems is shown in fig. 2.20, and under these conditions the beam would have a factor of safety against lateral buckling approximately equal to one as a train passed overhead. It is therefore not surprising that the bridge collapsed. The failure probably began with the quarter point fracture along the line of weakness created by the holes cast in the compression flange for the proposed ornaments. Whereas with a fracture initiated in the middle third the wrought iron strapping might have saved the bridge, it could do nothing to prevent this type of failure being catastrophic (fig. 2.21). The onset of this mode of collapse would be aggravated by the manner in which the train and deck loads were transmitted to the girders with a twisting component.

The Dee Bridge was more susceptible to this type of buckling failure

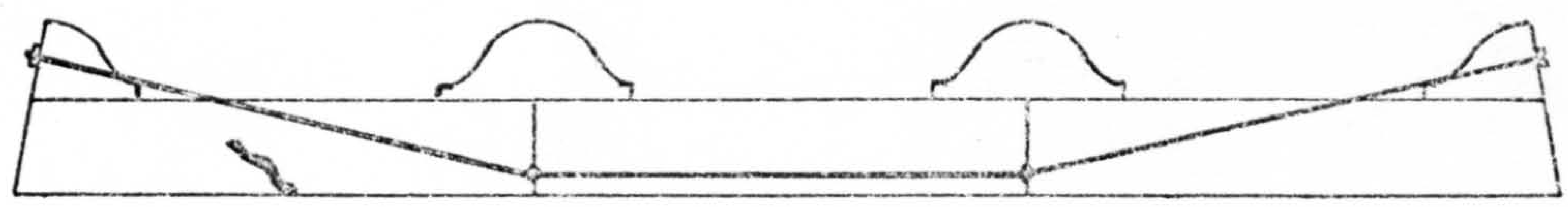

Fig 2.2I. Inability of tension rods to prevent failure in the end portions of a trussed girder.

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Fig 2.22. Trussed girder with tie-backs. (Engineer, J. Hawkshaw.)

than earlier structures because of the tensioning of the rods. Another unfavourable feature was the lack of continuous ties from span to span which Stephenson and others had used in earlier structures such as the R. Ouse Bridge at York and the Rye House Bridge on the Eastern Counties Railway (Table 2.2. and fig. 2.22).

2.6. The Accident and Contemporary Practice

The real cause of the accident was outside the everyday experience of most of the contemporary engineers. Those who spoke at the Inquest seem to have been more or less completely unaware that anything had been done which required more data than that supplied by Hodgkinson's formula. Only Kennedy referred to some model experiments where beams with a small upper flange had failed by buckling.

Similar attitudes prevailed at a discussion held at the Institution of Civil Engineers in April 1847, just one month before the Dee Bridge collapse.[34] This meeting was convened following the failure of a trussed girder in a Manchester Cotton mill (fig. 2.23) and several leading engineers attended. A descriptive paper was read by William Fairbairn and Messrs Farey, Bidder, Vignoles and Stephenson spoke in the discussion.

Some very odd statements were made and an impartial observer would have seen that so many different opinions could only conceal a general lack of understanding. Unfortunately there was no formal mechanism for summarising a discussion in this way, although such a conclusion might just conceivably have led to a reappraisal in time to save the Dee Bridge.

The beam in the mill was loaded in two distinct ways. Floor loads (including a component from vibrating machinery) were transmitted from the upper storey to the lower flange via shallow brick arches and with scant regard for shearing action (fig.2.24). Roof loads passed down the column A into the beam via a saddle resting on the compression flange. The stress in the column varied as the water level in the roof reservoir rose and fell.

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Fig 2.23.

Fig 2.23. Trussed girder in Messrs Gray Cotton mill, Manchester.
Fig 2.24. Balanced loading of girders in mill-work. Masonry arches and infill stabilise girders.

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Fig 2.24

Discussion of the collapse dealt mainly with the action of the wrought iron ties and the mechanism of failure. Fairbairn gave a figure for the amount of load that would be taken by the trussing, but it is highly unlikely that this was founded on mathematical reasoning because he was notoriously bad at even the simplest calculations.[35] Other easily verified mistakes in the paper corroborate this view - for example there is an incorrect statement about the distribution of roof loads and an estimate of the beam strength based on Hodgkinson's formula which is immediately contradicted by a remark that the beam probably buckled.

Stephenson agreed with Fairbairn, but did not quantify the distribution of load between beam and trussing.[36] He made the mistake of assuming that the ties could be applied without increasing the size of the compression flange required for an unassisted beam.

Other engineers thought the trussing was principally useful in preventing total collapse in the event of the cast iron fracturing in the tension zone (fig. 2.9). This was true in the earliest structures where the trussing was so flimsy as to be insufficient to do more than hold the cast iron parts together after a fracture. Messrs Bidder and Vignoles, two engineers who built some of the first trussed girder bridges were among those who supported this idea.

John Farey, an engineer celebrated in his own day for his inventiveness and for his work on steam engines in particular[37] set himself against the formidable proponents of these opposing views. He saw precisely what was wrong with the beam and his remarks could have been equally well applied to any trussed girder with ties ending above the neutral axis of the cast iron beam. To use his own words,[38] "The upper flange of the lower iron beam was obviously deficient in width to render it secure against lateral deviation from its horizontal straight line. The wrought iron truss rods which were applied to each side of the cast iron beams would have the effect of resolving part of the downward pressure exerted by the vertical column, into a force of compression acting in the direction of the length of the cast iron beam ...". Furthermore he could see how this could lead to failure even if he could not calculate the buckling load: "... The thin upper edge would be likely to be crushed, and to yield laterally to the compression caused by the action of the truss rods ...".

Later he pointed out that this type of failure was specially likely in this case because the roof load was transmitted onto a saddle resting on the top flange of the trussed beam. Any imperfections in the straightness of this flange would diminish the strength of the girder still further.

The issues raised by the contradictory views expressed in the discussion were not taken up because no one had a statutory duty to make an investigation. Most of the engineers mentioned above were engaged in the frenzied activity of railway building that gripped Britain in the late 1840s, and had very little time for researching. No establishments existed to undertake specialised studies, and only a few men like Eaton Hodgkinson worked solely on engineering problems.

The Government of the day had no power to control the practices adopted by engineers, and even if it had it would have been very unlikely to detect dangerous procedures because of the amount of work going on. Nevertheless the fact that cannot be denied is that there was no unanimous opinion about the cause of the collapse. This pinpointed the need for some provision to be made to stimulate and assist research even before the tragic circumstances of the Dee Bridge accident made the work imperative.

2.7 The Accident in Perspective

Perhaps pressure of work is an excuse which lets Government and engineers off their responsibility for the Dee Bridge collapse too lightly, because after all when the accident had occurred a thoroughly efficient Royal Commission was set up extremely quickly, which called on nearly all the leading expertise for information related to its investigations. Brunel, Stephenson, Locke, the Barlows, the Cubitts, Fairbairn, Fox and Raistrick all gave evidence and distinguished academics such as Stokes and Hodgkinson conducted experiments.

Since these engineers could find time for research and discussion when they considered it necessary, the view that the successful accomplishment of much railway work had made them complacent about their practices seems reasonable. The use of Hodgkinson's beam formula illustrates this point. This rule was published in 1835 and was supposed to give an estimate of the failure load for a centrally loaded simply supported cast iron beam.[39]

From tests on models up to 3m long Hodgkinson deduced that

$$W = \frac{c.A_b.d.}{l}$$

W = centrally applied breaking load (tons)
A_b = area of the bottom flange (in^2)
d = depth of the beam (in)
l = length of span (in)
c = casting constant: 25 for beams cast lying on their side; 26 for beams cast erect. (ton/in^2)

No area was specified for the upper flange but later work[40] established that for an economical section the area of the tension flange should be 6 times that of the compression flange, the difference in size being a reflection of the permissible stresses in tension and compression resulting from the notorious unreliability of cast iron in tension. In millwork potential instability was counteracted by the manner in which loads were transmitted to the girders (fig.2.24) but in bridges, where rather differentcconditions applied and where the beams were often loaded eccentrically (fig. 2.13) designers did not always see that it was desirable to provide additional material in the compression flange.

Certain other factors made real beams less strong than they should have been. Designers often assumed that a uniformly distributed load had the same effect as a central point load of half the total magnitude, but of course this practice under estimated the total magnitude of bending stresses. Another and more important underestimate of strength arose from the fact that whereas the models could be made more or less free from casting imperfections, real beams, and especially the longest girders, could never be made absolutely straight or free from a certain variability of texture caused by uneven cooling in the moulds. It is significant that Simmons observed in his report on the Dee accident that one of the girders had a distortion of 75 mm (3") from the straight line.[41]

The addition of trussing rods was not envisaged by Hodgkinson during the initial studies.[42] As has been mentioned, when they were first introduced the idea was merely to safeguard the main beams from brittle fracture, but in later work Stephenson and others tried to use them for prestressing.[43,44]. This would have been a good idea if the ties had been kept below the neutral axis of the beam, but in practice neither the positioning of the rods nor the size of compression flange required was properly studied. The use of a three link system physically connected to the cast iron gidrers at the third points of the span was another anomalous feature of the design as adapted for prestressing. Here there was insufficient provision for movement at the connection, so no forces could develop in the horizontal links. After the Dee Bridge accident, Stephenson very soon realised his mistakes and modified the works he had in hand to fulfil these conditions (figs. 2.25 & 2.26).

Taken together these remarks suggest that a continuous review of the development of trussed girders might have forestalled the disaster situation in which design information was applied outside the range of its validity. For example, a note of the basis of Hodgkinson's formula circulated among designers with a brief history of its application to trussed girder bridges would have shown that the original conditions (static loads on small beams) were very different from the railway loads and composite action of the latest bridges.

Better still, a graphical and tabular demonstration of the use of the basic design parameters such as that collected from contemporary sources and presented in table 2.2 and figs. 2.16 and 2.27 would have given a clear and quantitative indication of how far the original data had been extrapolated and a rational assessment of the risk being taken in pushing development still further.

Take, for example, the use of Hodgkinson's formula. Using the assumption made by the early railway engineers that the dead and live loads together constituted a uniformly distributed load, q, of about 1 - 1.5 tons/foot run, the total load carried by each girder of an

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Fig 2.25. River Ombrone railway bridge.

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Fig 2.26. River Arno railway bridge.

ordinary bridge (span L) would be $\frac{1}{2}.q.L.$ Since for normal purposes a uniform load could apparently be taken as generating the same bending forces as a central point load of half the total magnitude, the equivalent central breaking weight for such a beam would be $W = \frac{q.L}{4} = \frac{cA_bd}{L}$ from Hodgkinson's formula. It would therefore be reasonable to suppose from the initial assumption that $\frac{q}{4c} = \frac{A_b d}{L^2}$ = constant, i.e. that there would be a linear relationship between A_bd and L^2 for the range of spans considered. If these quantities are plotted from Table 2.2, it can be seen that Stephenson's bridges at Chester and Stockton were seriously understrength in terms of contemporary practice (fig. 2.27 - the various lines represent different safety margins). The Chester bridge of course collapsed, and a search of Stephenson's correspondence shows that after the accident he was very worried about the security of the bridge at Stockton.[45] This weakness could, however, have been detected if a continuous record of changes in current design parameters had been kept. The fact that most of the bridges in table 2.2. were constructed between 1844 and 1847 pinpoints the need to up date such a record regularly.

Unfortunately this information was not available in contemporary books or periodicals. In fact the statistics that have been collected were largely derived from the survey of iron bridges carried out by the Commissioners of Railways immediately after the Dee Bridge collapse, which demonstrates the feasibility of carrying out such a survey even at a time of enormous pressure of work. Of the 226 Railway Companies circulated with a letter asking for details of iron bridges on their lines 163 replies were received presenting a fascinating picture of railway practice at the time. Table 2.1 is a general breakdown of the statistics giving a perspective view, not available before the accident, of the

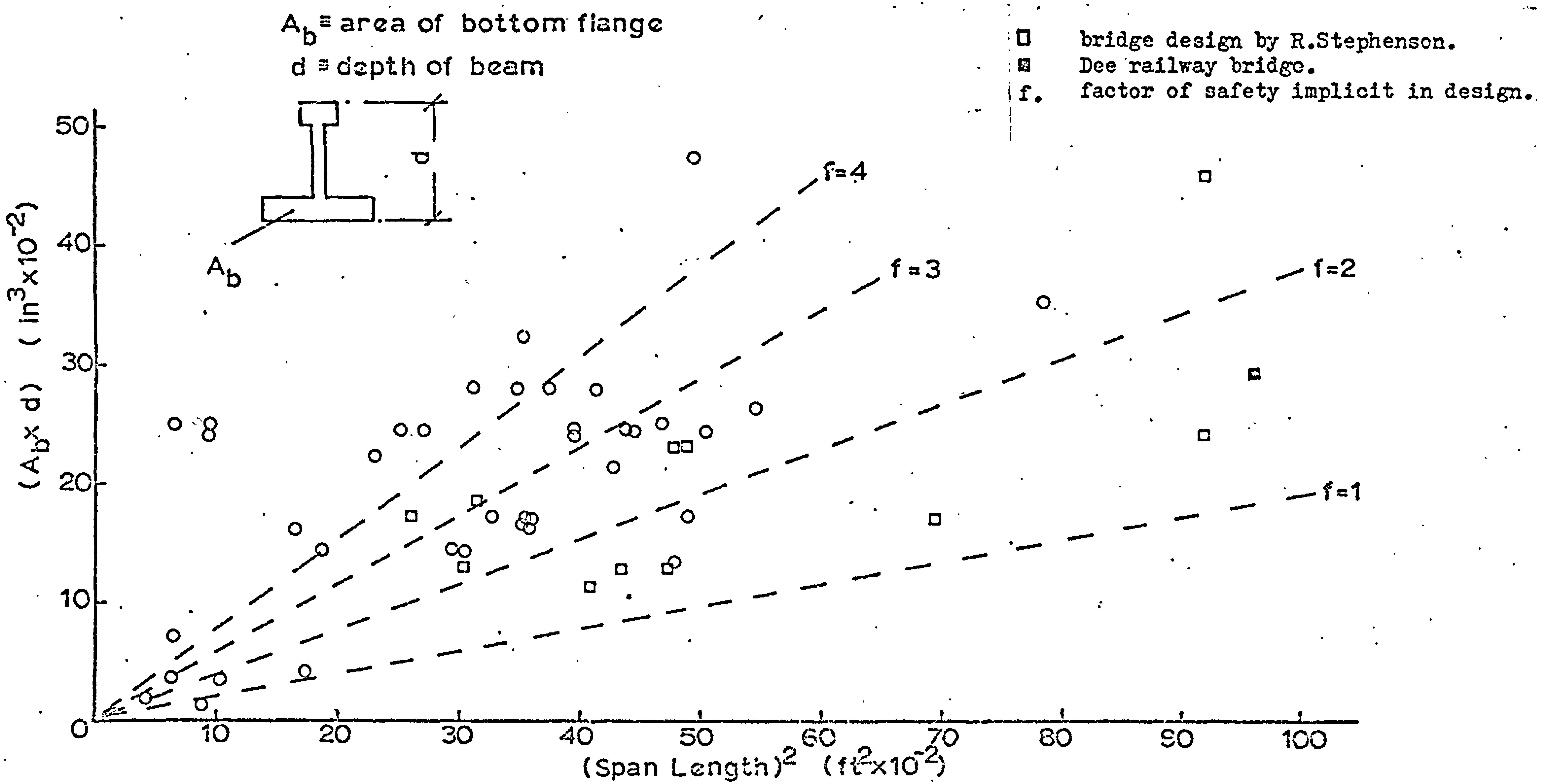

Fig 2.27. Graph showing the relative safety of trussed girder bridges

iron railway bridges constructed throughout the country, while Table 2.2. contains rather more detailed information about trussed cast iron girders. From Table 2.1 it is clear that most engineers considered the trussed girder unsuitable for spans greater than 20 m (70 ft), the preferred alternatives being cast iron arches, bow and string girders, and tubular girders on the Fairbairn pattern. Lattice and truss girders, later to become the dominant form for spans of 25-60 m (80-200 ft) had hardly been applied at all by this date. The main lesson to be drawn from this table is that if Stephenson had been faced with this body of opinion at the time of the design, he might have been more careful about his extrapolation of the trussed girder principle.

Enough has now been said to show that in engineering terms the collapse of the Dee Bridge was not just a chance occurrence. but the natural outcome of a period of change and development. It is also interesting to try and quantify the total effect of neglecting the research function during the years of railway development up to 1847. This affected not only the trussed girder bridges, but also all the under-line structures in which the fundamentally unsuitable cast iron material was used.

Broadly speaking, the history of the Dee Bridge after the collapse was repeated with all the other sixty odd trussed girders already built or under construction. These were strengthened, usually by bolting additional pieces on to the original castings (fig 2.28). This gave an outward appearance of solidity, but in reality it must have been very difficult to obtain an accurate fit and even bearing for the new pieces. Engineers obviously realised that this was only a face saving

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Fig 2.28. Compound trussed girder
with strengthened compression flange.

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Fig 2.29. Chepstow bridge, I.K.Brunel,
engineer.

measure because apart from the much larger scale Chepstow railway bridge built by I K Brunel in 1852 (fig 2.29) no new structures using the principle were designed. As soon as railway loads began to increase with the development of heavier locomotives the opportunity was taken to remove many of the old structures and replace them with something more satisfactory.

Some idea of the cost of replacing these indifferent structures can be derived from information given by G D Dempsey in his contemporary essay.[46] Writing in about 1850 he said that the cost of ironwork for an 18.3 m (60') trussed girder was £1,500. This means that the absolute minimum cost of replacing all sixty trussed girder bridges must have been £90-100,000 at 1850s prices. Temporary closures, propping and strengthening probably more than doubled this.[47]

Table 2.1 shows that at the time of the Dee Bridge collapse there was a total of some 1,400 cast iron railway bridges in service. Many of these were replaced within a few years by one of the wrought iron forms developed after the publication of the Royal Commission report in 1849.had pointed out some of the deficiencies of cast iron. Taking this into acoount the final bill for neglecting research during the early part of the railway age can fairly be said to be about £1.5 m from this source alone.

Mention was made earlier of the fact that at the time of the Dee Bridge collapse engineering work on new railways was being carried out on an unprecedented scale. It is interesting to look a little more closely at this so-called railway mania to see how the whole climate in which work was carried out may have affected the adoption of trussed girder and other cast iron structures for railway purposes.

To turn back a few years, in the 1830s a great deal of engineering work was successfully accomplished, financed by the solid capital of the towns and industries that were the principal beneficiaries of the lines constructed. Many of these were built on a truly monumental scale - the Great Western main line from London to Bristol being the best example, but there were others including the Liverpool-Manchester and London-Birmingham Railways. The cost of these projects was enormous, and to start with investors found that the return on their capital was very poor indeed. This was partly their own fault because the fares charged frequently reflected the capital invested and were consequently very high, but there were other factors as well, such as an acute trade recession.

As a result, when the 1840s opened railways were regarded as a poor investment and no new lines were proposed between 1840 and 1842. The economic stagnation lasted through these years but at the beginning of 1843 prospects began to look a good deal healthier. Railway business became more attractive because fares had been reduced, and because some of the excesses of surplus manpower and unnecessarily lavish equipment had been pruned away during the years of austerity.

However, the principal factor that led to a sudden revival of railway building was that a lot of investment capital, formerly diverted to a miscellany of overseas interests suddenly became available in the City. At the beginning of 1844 there were only 3280 km (2050 miles) of railway in Britain with another 640 km (400 miles) under construction, but as interest in railway shares caught on in the investment world applications for new lines poured in from capitalists and speculators alike so that at the beginning of the Parliamentary session the Government suddenly found 1440 km (900 miles) of proposals awaiting its consideration. If this was not enough, the Board of Trade steered a bill through

Parliament reducing the capital necessarily deposited before construction of a railway could commence from ten to five percent. The idea of the bill was to encourage construction work and reduce the widespread unemployment associated with the recession.

The new lines had a variety of purposes. Although most large towns had a railway before 1844, there were many useful journeys that could only be accomplished by roundabout routes, so some of the new lines provided direct connections. Other railways were projected to connect outlying ports to the system. Among these was the Chester & Holyhead line which connected an existing main line from London with the shipping port for Ireland.

In 1845 the railway situation ran wild; people could see that a stake in what was emerging as the dominent form of transport was an excellent investment so there was a rush to buy shares even at a premium. Many new lines were projected, and gullible folk were often induced to buy stakes in schemes regardless of the traffic usefulness or constructional difficulties of their railway. During the year Parliament granted powers to construct 4600 km (2880 miles) of new line at an expenditure of £44m.

The mania continued throughout 1846, and the Government sanctioned £120m of applications out of a total of £389m. Some of these were sound but others were justified on very shaky evidence. For example, some lines paralleling existing railways were authorised merely to allow the public the supposed benefit of unrestricted competition.

These were dizzy manipulations of finance, frequently unrelated to the needs of the country as a whole. There were repercussions everywhere, and very naturally these were felt in the Civil Engineering industry as strongly as anywhere else. The fact that there was very little work available for railway engineers between 1838 and 1843 meant that even consulting practices as famous as Robert Stephenson's at 24 Great George Street found themselves seriously lacking in trained assistants when the rush began. These firms were extremely overworked, and to make matters worse their senior staff were diverted from design work to the tedious business of preparing and presenting interminable evidence to Parliamentary Committees.

The volume of work undertaken by these men, committees and engineers aloke, was quite staggering and took its toll on the quality of intellectual judgements. Committees authorised futile schemes and engineers had no time to optimise and check each design systematically. Under these circumstances errors such as the misuse of cast iron were bound to occur.

This was a very trying background for the engineers to work against, so the question that naturally arises is whether the railway mania could have been averted. The answer, as in other similar circum-stamces, is yes it could, but it was not because it was more expedient for the financial and political interests to let it happen. An [illegible] unrepeatable opportunity for creating a railway policy was missed in the early 1840's, and when, in 1844, a proposal eventually came outlining the principles of a nationalised rail system it really came too late. The measure met with bitter opposition in Parliament (reflecting the vested interests of members) which might not have been the case had it happened in the years when existing Railway Companies were facing serious problems.

2.8 Summary & Conclusions

This chapter can best be summarised by setting out the key facts relating to the Dee Bridge collapse in chronological order. First among these was the design principle applied to the main girders. This rule, known as Hodgkinson's formula, was devised in about 1830 to rationalise the design of simply supported cast iron beams loaded with a statically applied weight at mid span. It was widely used for proportioning girders in ironframed mills and factories which required spans of up to 10 m. In these structures the loading was usually applied symmetrically and in such a way as to stabilise the girders.

The rule was next applied to the design of built up beams and trussed girders where the span required was greater than could be obtained with a single casting. These girders were then introduced into railway bridges where a different class of loads had to be carried in a less satisfactory manner. In the earliest crossings designers played safe and treated the trussing as a safety mechanism only. The success and economy of these bridges led them to experiment with longer spans and latterly with an empirically devised type of pre-stressing but by this time the structures being built were very distant relatives of the type of girders originally envisaged by Hodgkinson. Nevertheless the same principle was applied.

The discussion that followed the collapse of a girder in a Manchester cotton mill showed that most designers did not realise that the principle had been extended in such a way as to render it invalid. Had this discussion been satisfactorily interpreted with reference to the original design data, or had there been a continuous review of developments such as that made immediately after the accident the trussed girder principle and even the whole usage of cast iron might have been questioned in time to save the Dee Bridge.

As things turned out the situation had to be remedied in the unfavourable circumstances of a public panic. A Royal Commission was hastily set up to investigate the problems and find practical solutions. This it did, in a meritorious if not all-embracing fashion, by conducting inquiries and by testing hypotheses with experiments.

After publication of the 1849 Report, the fuss died down and the matter slowly faded from memory. This however left some of the basic issues still undiscussed, notably the fact that something should have been done to prevent a recurrence of the situation whereby a period of unnoticed change and extrapolation led to a disaster. As will be shown, this has happened several times since the events of 1847.

In this case, to have broken the train of development before it resulted in a failure would have required the presentation of data such as that shown in Tables 2.1 and 2.2 and fig. 2.27 to designers as a background against which to take risks.

This can be pointed out in retrospect, but against the highly unsatisfactory economic conditions governing civil engineering work in the 1840s it was impractical to collect such data until a disaster made it imperative, especially as practically no-one was willing or able to carry out the necessary research.

3. The Tay Bridge

3. The Tay Bridge

3.1 History of the Undertaking

The theme running through these studies is that it should be impossible for any engineer to execute new work which goes outside the intended scope of the data and principles on which it is based. About thirty years after the drama of the Dee Bridge accident, the history of Sir Thomas Bouch's Tay Bridge provides another example of how a large and daring structure was built using design information which, although it had proved adequate for earlier smaller scale designs, was wholly insufficient for the design of a much larger project.

The bridge in question was the first really long high-level crossing of a storm lashed estuary (fig.3.1). The single track main line of the North British Railway was carried on a wrought iron superstructure 3.2 km long which reached a maximum height of 27 m above the water. There were 85 truss spans varying in length from 8 to 75 m; depth/span ratio was about $\frac{1}{8}$ and the width 3.8 m. The piers were of various types but groups of cast iron columns crudely braced with wrought iron predominated. Below water level these were carried down to the estuary floor via a masonry substructure. The total cost was £350,000 compared with £217,000 estimated in 1871 just before construction began.

The whole seven year construction period was dogged with every kind of difficulty. As with so many cut price projects the optimism of the initial proposals was not borne out in practice.

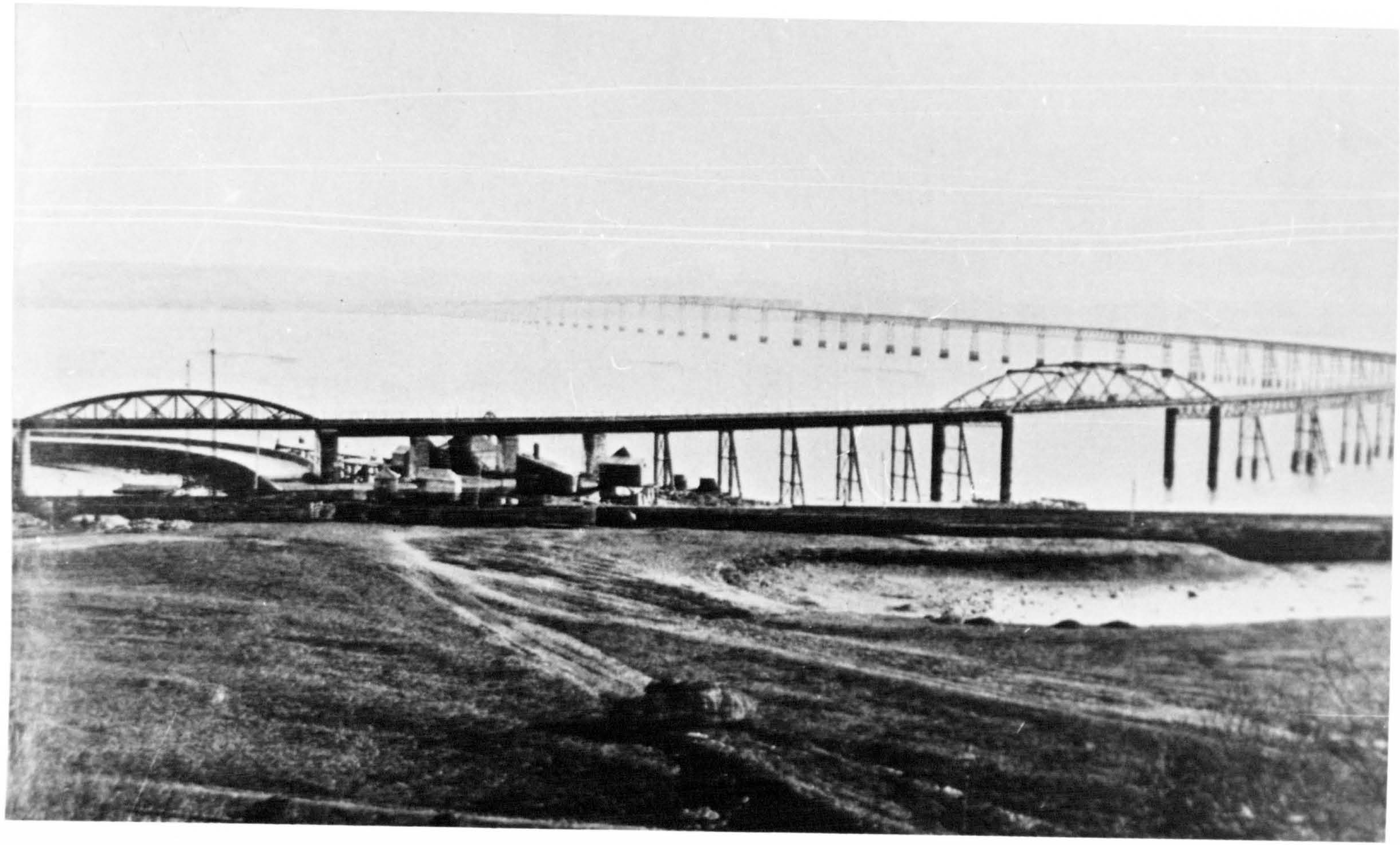

Fig 3.I. The first Tay railway bridge, seen from the north.

For example, although rock was believed to exist just below the sand layer on the estuary floor, it was soon found that the seemingly solid material was nothing more than a thin crust of conglomerates covering an indeterminate depth of mud and clay. This led to a decision to reduce the weight of the structure by abandoning the masonry towers originally intended in favour of cast/wrought iron substitutes. However, the underwater part of the piers was not altered, so the iron-work had to made do with the very restricted base area available. This gave the bridge a rather top heavy appearance. (fig.3.1).

On the contracting side matters were worse still. The original contractor, Charles de Bergue, died in the spring of 1873, and his heirs asked to be relieved of the contract, but not before it had been discovered that the tender price was hopelessly unrealistic, presumably because de Bergue had desperately needed the work. His firm was replaced by Hopkins, Gilkes and Company of Middlesborough, who fell into the trap of trying to save money by using poor quality materials. If they had been able to do this satisfactorily, the total saving on such an enormous structure would have been very great, but unfortunately the iron ore they used from mines at Cleveland was so poor and so full of impurities that it was well nigh impossible to make decent quality castings from it. Thus a lot of the ironwork in the piers bore little relation to the designs (the wrought iron for the spans was manufactured much more satisfactorily at Middlesborough). Matters were made still worse by the near intolerable working conditions in the specially built foundry at Wormit on the south side of the estuary, so all in all it is hardly surprising that the column castings were often of very uneven thickness and full of blowholes and cracks. The metal was so difficult to work that details such as bolt holes and lugs cast integrally with the columns were rarely formed accurately and 'cold shuts' (castings in which some of the metal sets before molten iron from a different route through the mould joins it) were not uncommon. Unfortunately, none of these matters came to light until after the accident, because there was no adequate provision for quality control during the construction period; indeed none of the supervisory staff had any experience of foundry work.

The bridge was inspected and passed fit for traffic in February 1878, and was officially opened on May 31st. Queen Victoria crossed it, Bouch was knighted, trains plied to and fro, and the North British Railway Company at last began to make money out of the undertaking.

However, no proper provision was made for maintenance and the inexperienced men retained to look after it made little or nothing of the excessive vibration of the metalwork and the considerable numbers of nuts, bolts and rivets that they found on the structure from time to time. One or two badly cracked columns were strapped with iron collars, and some bracing members were tightened with crude wedges, but by and large the structure was regarded as a success and was not unduly tampered with.

However, after nineteen months of successful operation the bridge came to an untimely end. On the night of December 28th, 1879, a severe gale blew down the estuary perpendicular to the bridge. The intensity increased towards 7 pm but no one thought of suspending traffic. At 7.13 a north bound train moved out across the estuary but on reaching the highest and most exposed of the through spans the wind blew it and a considerable portion of the bridge into the foaming waters below.

3.2 The Wind Loading Problem

A Court of Inquiry investigated the accident, and heard a great deal of evidence which uncovered the many defects of design and construction alluded to above. Some of the discoveries were so dramatic that they eclipsed the main issue, namely that even if everything else about the bridge had been perfect, it would still have been inadequate to resist wind forces.

Now whereas the many malpractices and inadequacies of the construction and maintenance period have been retold and discussed time and again,[1,2] the important matter of wind loads has never previously been fully researched. This chapter therefore sets out to discover whether the designer was really to blame for not providing adequate bracing for the structure.

It turns out that although some engineers had developed an empirical knowledge of wind forces by the early 1870s, there was no generally accepted scientific method for estimating the wind loads likely to be encountered. In this instance the chief engineer believed (or was led to believe) that wind pressure would not generate significant forces in the structure. He applied for information to a recognised authority on the subject and received an unfortunately worded reply, which, instead of presenting a clear picture of the limitations of existing knowledge, concealed many uncertainties and inaccuracies.

The underlying problem was that it was very difficult to assess the effect of wind on structures and particularly reticulated structures before the use of wind tunnel models was pioneered at the end of the nineteenth century. As a result, wind loading calculations were not normally carried out for structures completed before the Tay Bridge accident, although a system of horizontal bracing was always incorporated to assist load distribution etc.

When a design check was made for wind action the procedure adopted was to assume that a uniform pressure of wind acted on the exposed face of the bridge and that some fraction of this acted on the lee girders. The precise amount depended on an intuitive guess of the degree of shielding afforded by the windward members - an unsatisfactory procedure because designers did not understand effects such as drag and lift forces. To show how seriously wind forces were underestimated, fig.3.2 compares four loading cases. Fig. 3.2a is based on the wind pressure assumed in design - 485 $^{N}/m^2$ (10 lbs/ft^2). 3.2b shows the forces necessary to overturn the structure about the base of the lee pier. This calculation was made by Mr Henry Law who was commissioned by the Tay Bridge Inquiry to analyse the structure after the collapse. From the evidence of the wreckage, he showed that the windward columns were inadequately anchored to the pierheads, so he discounted the possibility of beam action in the towers. He also showed that this mode of failure was more likely than the overturning of the spans on the tops of the towers.[3]

Fig.3.2c shows the wind forces suggested by the Board of Trade rule which was established after the collapse. Although this was ridiculed as extremely conservative by engineers at the time, it is interesting to note that the total force and overturning moment generated by it is very considerably less than the corresponding quantities calculated from the 1972 wind loading code, CP3, ch.5 part 2 (fig. 3.2d).[4]

Why did Bouch choose such a low figure for the assumed loading? To answer this question it is important to understand that his main

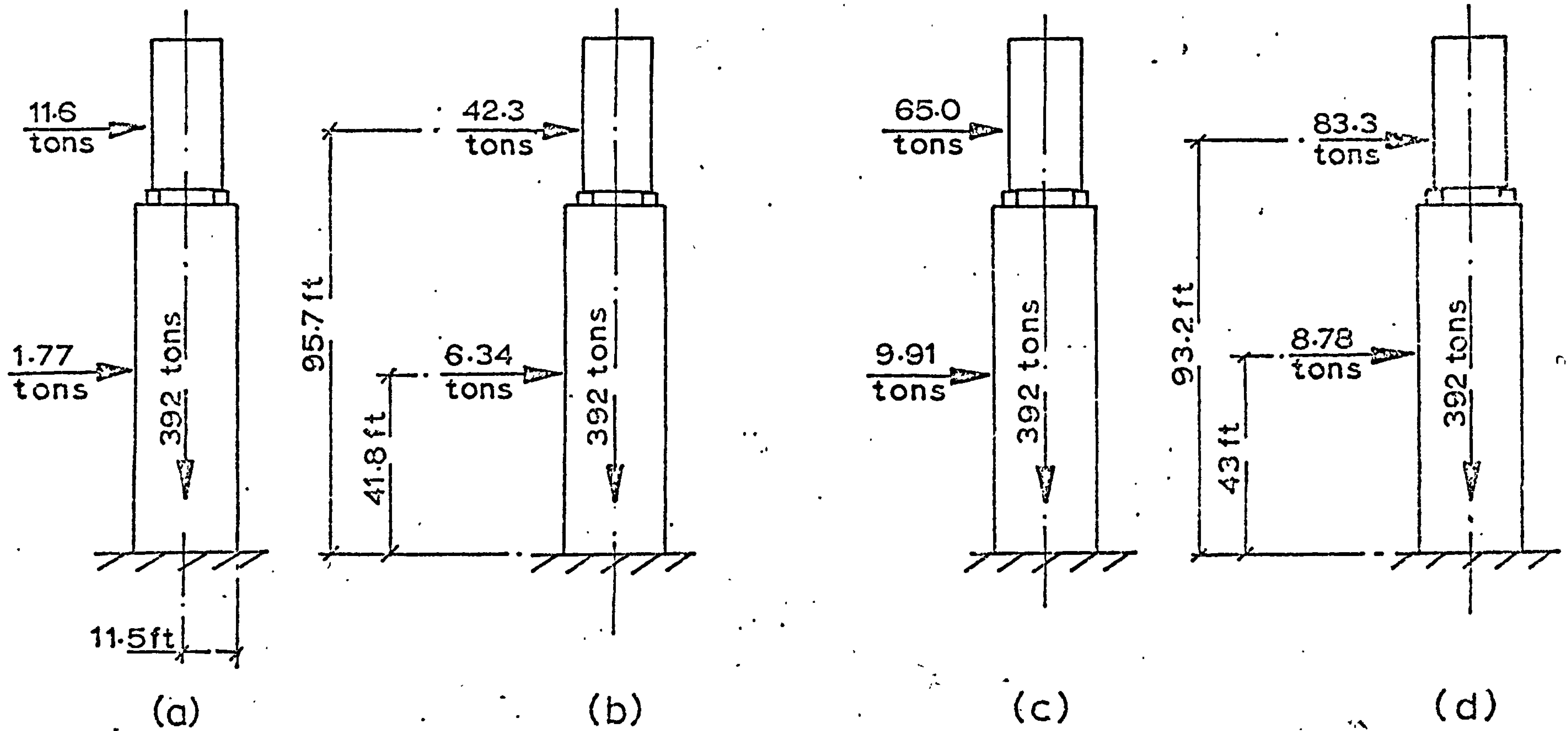

Fig 3.2. Wind forces as calculated by four different methods.

aim as a designer was to provide an adequate structure as cheaply as possible. Ample evidence of this is provided by his earlier works. In designing the Tay crossing he very rightly wanted to know what an adequate, but not excessive, provision for wind forces would be. Consequently he looked around for the best advice available.

The normal source of information about wind pressure was the table communicated to the Royal Society by Smeaton in 1759 [5] (Table 3.1), but this had long been acknowledged as over-conservative and inaccurate by most authorities. (Prof. Rankine, author of the well-known Manual of Civil Engineering [6] being the best known exception; he thought that the design wind load should be the max.gust pressure ever recorded, this being 2660 $^N/m^2$ at Glasgow Observatory. For reasons to be discussed later this was not satisfactory.)

The values given in table 3.1 were derived from experiments using the rig sketched in fig. 3.3 [7] and the records of various velocity anemometers. In the experiments the test plate A travelled round a 9.1 m orbit at velocities of up to 1.05 m/sec. It will be appreciated that the resistance experienced by the plate was much greater than the stagnation pressure of the air stream because eddying round the edges of the plate created a partial vacuum on the lee surface. Values of pressure corresponding to higher wind velocities were extrpolated from the experimental results by assuming that pressure was proportional to (1) the area exposed and (2) square of the velocity. Thus for most practical cases the air resistance corresponding to an assumed wind velocity was considerably over-estimated.

Edwin Clark commented on this data in his excellent book about the Britannia and Conway tubular bridges. [8] He dismissed Smeaton's figure of 2400 $^N/m^2$ uniformly distributed as "a much overrated value for hurricane loading in the British Isles". He said later

TABLE 3.1

"containing the velocity and force of winds, according to their common appellations"

Source: Phil.Trans. Royal Soc. vol 51, 1759, p.164

Velocity of the Wind		Perpendicular force on one foot area in pounds avoirdupois [& metric equivs.]	Common appellations of the force of winds
Miles in one hour	Feet in one second		
1	1.47	0.005	Hardly perceptible
2	2.93	0.020	Just perceptible
3	4.40	0.044	" " "
4	5.87	0.079	Gentle, pleasant wind
5	7.33	0.123	" " "
10	14.67	0.492	Pleasant, brisk gale
15	22.00	1.107	" " "
20	29.34	1.968	Very brisk
25	36.67	3.075	" "
30	44.01	4.429	High winds
35	51.34	6.027	" "
40	58.68	7.873	Very high
45	66.01	9.963	" "
50	73.35	12.300 595N/m^2	A storm or tempest
60	88.02	17.715 860N/m^2	A great storm
80	117.36	31.490 1530N/m^2	An hurricane
100	146.20	49.200 2380N/m^2	An hurricane that tears up trees, carries buildings before it, etc.

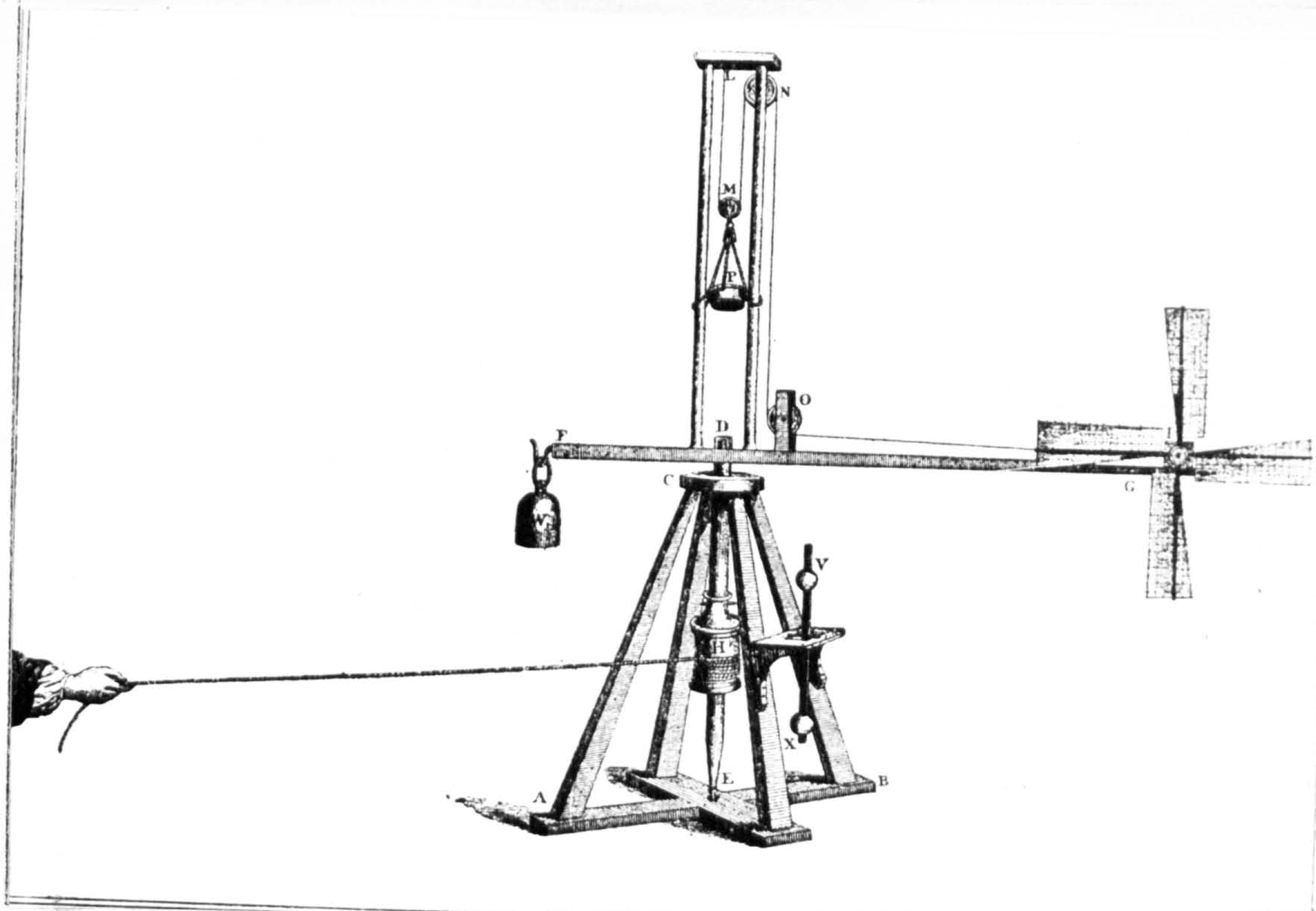

Fig 3.3. Smeaton's test rig. In the experiment on wind pressure, the windmill sails were replaced by a board of 3 sq.ft. surface area.

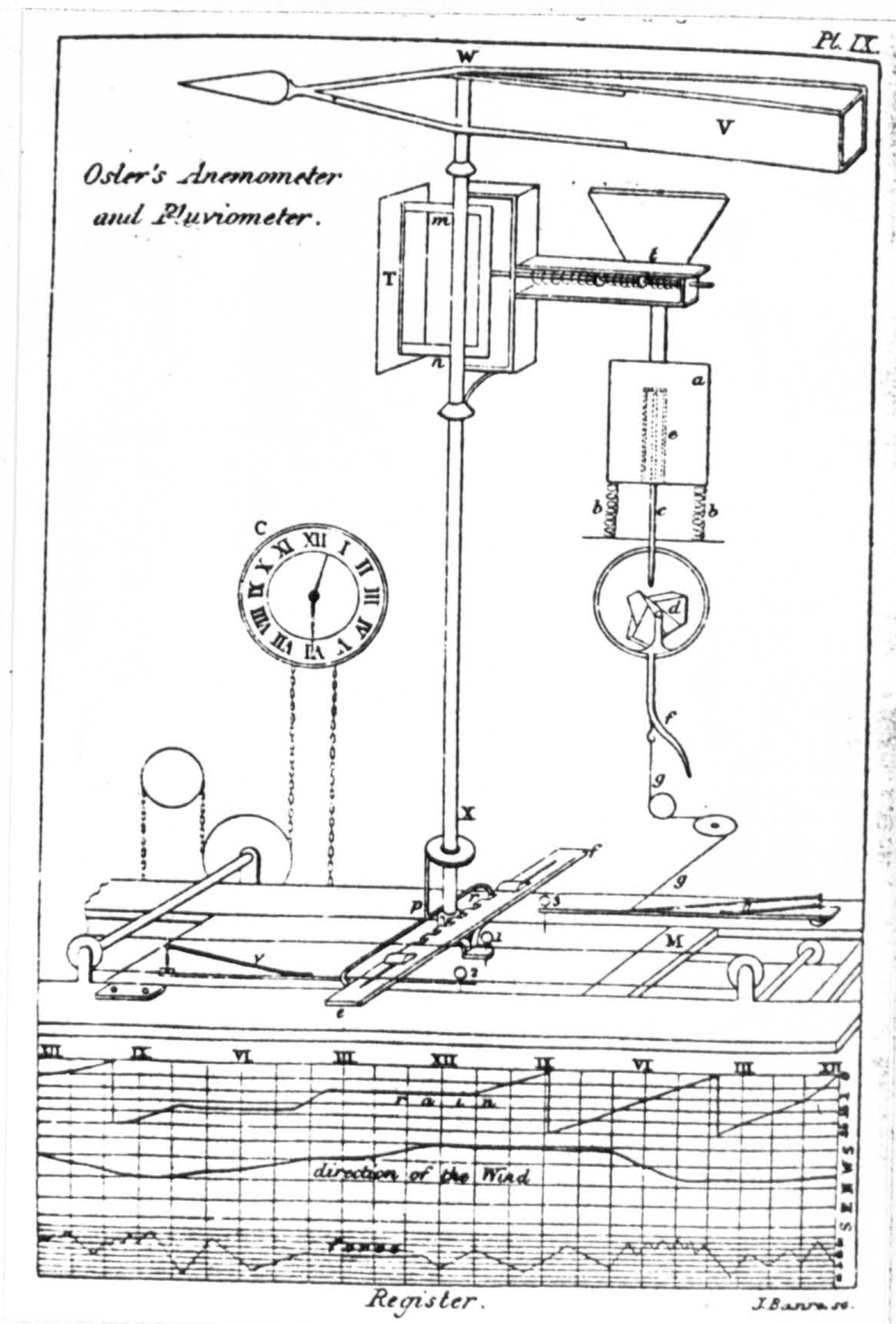

Fig 3.4. Osler's pressure plate anemometer. On the graphs, one horizontal division represented an hour, so the instrument was not able to record short duration gusts.

that "a violent storm exerts a force of about 20 pounds on every square foot (= 970$^{N}/m^{2}$) of surface exposed to its direct action."

Bouch was dissatisfied with this information and therefore referred to a letter he had received in connection with another of his projects, the Forth Suspension Bridge which he proposed in 1873. That plan was so novel that the North British Railway who commissioned it had the designs scrutinised by a committee of experts (W.H. Barlow, John Hawkshaw, Dr Pole, Messrs Bidder & T E Harrison). They questioned the validity of Smeaton's table of wind pressure and applied to the Astronomer Royal (Sir George Airy) for an up to date opinion. Airy was consulted because he was responsible for the maintenance of meteorological records at a number of stations up and down the country. His reply (which the committee endorsed) was received in time for the Tay Bridge superstructure to be modified if necessary - indeed for other reasons the ironwork was much altered after this date.

Airy wrote, "My dear Sir, - I have considered carefully the proposal of constructing the suspension bridge over the Forth for carrying a railway in two spans of about 1600 ft (490m) each, upon which I understand you wish that I should give you my opinion. First upon the liability of the bridge to suffer from the action of the wind; (secondly upon any points in the construction which might occur to me. I will advert here to both subjects.) We know that upon very limited surfaces, and for very limited times, the pressure of the wind does amount to 40 lbs per square foot (1940$^{N}/m^{2}$) or in Scotland probably to more. So far as I am aware our positive knowledge, as derived from instrumental record goes no further. But in studying the registers it is impossible not to see that those high pressures are momentary and it seems probable that they arise from some irregular whirlings of the air which extend to no great distance. I should say certainly to no distance comparable with the dimensions of the proposed bridge, and that the fairest estimate of the pressures on the entire bridge would be formed by taking the mean of the recorded pressures at one point of space for a moderate extent of time as representing the mean pressure upon a moderate extent of space at one instant of time. Adopting this consideration, I think we may say that the greatest wind pressure to which a plane surface like that of the bridge will be subjected on its whole extent, is ten pounds per square foot (485 N/m^2)." [9]

Bearing in mind that the most exposed sections of the Tay Bridge consisted of continuous girders more than 305 m long, Bouch being dissatisfied with other sources of information felt justified in assuming that this advice would apply to the bridge actually in course of construction.[10] Moreover the actual magnitude of the wind force had been accepted by five very eminent engineers. It is doubtful that he would have been given better advice elsewhere; as late as 1879 the official Board of Trade view was that no provision was necessary for wind forces in truss bridges of less than 60 m span.[11]

By comparing this information with that contained in the modern Code of Practice it is clear that Sir Thomas's information was grossly inaccurate. Perhaps the worse feature of the report quoted is that it did not give sufficient quantitative information for the engineer to judge whether or not the data covered his problem adequately. Reading between the lines, there is a certain vagueness about the document. What precisely did the Astronomer Royal mean by very limited surfaces and very limited times? What did he really know about conditions in Scotland? Did his instrumental records really justify his conclusions? These matters were all investigated by the Court of Inquiry and it is worth setting out what the Commissioners discovered.

3.2.1 Gusts

On the subject of the length and extent of gusts Airy really knew absolutely nothing. Cross-examined by counsel for Sir Thomas Bouch his opinion was decidedly unformed:-[12]

16122 Q. I would like to ask your opinion upon this question. Do you believe that any such pressure as 40 or 50 lbs to the square foot (1940-2400 $^N/m^2$) ever obtains over so large an extent of surface as is represented by a span of the Tay Bridge - that is to say a length of 245 ft (75 m), and a height, including the piers, to the top of the girders of 100 ft (30 m)?

A. I have no means of knowing.

16123 Q. What is your judgement upon the point?

A. I can conceive the course of the wind on these occasions to be almost as irregular as the course of streams in a river; there are some parts in which the pressure will much exceed that in others, in fact, the space through which the greatest force prevails is limited, I imagine.

16124 Q. Although you have observations which indicate so high a pressure as 40 or even 50 lbs to the foot (1940-2400 $^N/m^2$) at a particular point, is it not, in your opinion, highly improbable that any such pressure has ever obtained simultaneously over a large area?

A. I really cannot say that I have any very definite opinion about that.

These answers boil down to the fact that the only information Airy had on the important issue of gust loading was the records of the individual Osler anemometers (fig. 3.4) maintained by just three of the meagre national network of Meteorological Stations. The instruments in question gave a crude measure of peak wind forces experienced by a small flat plate, but the movement of the recording paper was so slow that no information was available concerning the duration of gusts. None of them were kept in Scotland anyway.[13]

Any other direct measure of wind pressure that Airy might have received came from anemometers of the Lind type (fig.3.5). This instrument (basically the same as a modern Pitot tube) was invented by James Lind in 1775[14] and was often used on sailing ships during the 19th century. Here, therefore, the idea was to record average wind pressure as and when required so that the speed of the ship might be estimated from a rule of thumb knowledge of its normal performance. As a result the instrument was not supplied with a complicated recording system. It gave no indication of the magnitude or duration of gust loading because a contraction at the bottom of the U tube damped out sudden oscillations of the fluid.

The instruments for measuring wind velocity were no more helpful; Robinson's cup anemometer (fig.3.6) was widely used, but principally for recording the hourly run of wind. As a result the only mechanism attached to the rotating cups was a revolution counter geared to take account of the difference between the velocity of the wind and the speed at which the cups spun round. (In fact, the relation between these quantities was not well established and varied from instrument to instrument.)

The Inquiry called for the expert opinion of Mr R H Scott (Secretary of the Meteorological Council) and he said that records from these instruments were susceptible to considerable inaccuracy for periods of five minutes or less, so they also were useless for measuring gust duration.[15] Another snag was that even if the wind velocity could be

Fig 3.5. Lind's portable wind gauge, principally used on board sailing ships.

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Fig 3.6.
Robinson's cup
anemometer.

FIG. 32.

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Fig 3.7. Variation of wind velocity with height gauged accurately there was considerable doubt about the relation between this quantity and the pressure experienced by actual structural components.

The fact that none of these anemometers were fitted with sensitive recording mechanisms reflects a lack of interest in gust loading rather than insuperable technical difficulties. Dr Robinson, for example, was able to fit a recorder with an enlarged time scale to his instrument as soon as the Court of Inquiry asked him to do so.[16]

Another aspect of the problem that was not properly investigated before the accident was the area over which the gust loads acted. This would have been possible using a network of anemometers each supplied with a clock and sensitive recording mechanism, but the impracticalities of the process meant that experiments on the spatial extent of gusts were not conducted before the introduction of electrically-coupled instruments.

3.2.2. Winds on Site

Thomas Stevenson, however, did use a mechanical method to tackle a further problem of which Airy was completely unaware. Using six cup anemometers on a pole fifty feet high he carried out an investigation of the variation of wind velocity with height and showed that the velocity increased parabolically above a boundary layer wherein the increase was irregular owing to the roughness of the ground surface (fig.3.7).[17] His interest in the matter, of course, was in connection with the family engineering practice which specialised in lighthouses. Founded by Thomas's father, Robert, the firm was largely responsible for the network of lights built up round the hostile Scottish coats.[18] To illustrate the importance of this work (which was carried out ln 1878) a gale registering 44m/s 6.1 m above ground was found to correspond to 61 m/s at 30.5 m while the associated value of wind pressure rose by 88%.

Another weakness in the Astronomer Royal's report was that it only just touched on the question of how conditions vary from site to site. Most of Airy's information was gathered from instruments kept in and around London where the ground formation and surface features are comparatively sheltered. Today's designers use a wind speed of 38 m/s in that area whereas 53 m/s or more is allowed for exposed sites in Scotland.

3.2.3. Conclusion & Summary

Perhaps these paragraphs seem unduly critical of one man. If this is the case it should be remembered that Airy's opinions were endorsed by some of the best talents of the day. Moreover, contemporary authors rarely mentioned the subject - Rankine dismissed it in the single paragraph cited above,[19] while Humber[20] & Stephenson[21] did not discuss it at all. Airy's letter was also used as the basis of questions put to two expert witnesses called by the Inquiry to discuss wind loading, neither of whom (Prof. G G Stokes,[22] Mr R H Scott)[23] was able to offer more definitive information so the report can fairly be taken to represent the general opinion of the day.

In summary perhaps it is worth reiterating the lesson to be learned from Airy's report. Here was a statement based on scanty information in which uncertainties and inaccuracies were casually glossed over. At best the meteorologist should have said that very little useful data was actually available and that an on site investigation was really necessary. Failing this he ought to have referred the engineers to the source material so that they could form their own opinions on the doubts surrounding the subject.

3.3 How the Disaster might have been avoided

3.3.1 Introduction

The misunderstanding that arose between Bouch and Airy over the report discussed above was a breakdown in communication at a personal level. It was particularly unfortunate that the engineer seems to have relied solely on this source of information and it is interesting to look more generally at the work of his contemporaries to see if it might have been possible at that time for the Profession as a whole to have devised a system for avoiding the situation in which an important problem remained unrecognised until a disaster occurred.

Among the leading engineers of the later 19th century Benjamin Baker (designer of the Forth Railway Bridge) was making a name for himself during the 1860s. During this decade he began to realise that wind loads might prove significant in future generations of structures. In the course of his work he made notes of wind damage and later made simple calculations of the uniform wind pressure that would have caused the accidents. By studying objects as varied as stone walls and factory chimneys he developed a pretty clear understanding of the basic nature and magnitude of wind loads: his records suggested 1360 N/m^2 as an upper bound for the pressure on large structures.[24] After the Tay Bridge collapse he commended the collection of this type of straightforward data to his colleagues.[25] His observations led him to appreciate the special value of collecting information relating to actual structures. In the course of his researches, he found that the eminent American bridge engineer, Charles Shaler Smith had collected and published records of truss bridges destroyed by wind on that continent.[26] These accidents (Table 3.2) led Smith to conclude that 1450 N/m^2 was a sensible basic allowance for wind forces.

3.3.2 Practices in America

Baker's work suggests two of the important concepts regarding the prediction of accidents. Firstly he established the usefulness of recording failures that were not fully explained by existing knowledge, but his report of American experiences was really more important because it pointed out the value of keeping track of developments abroad.

In this particular case Bouch and the other British engineers could have learnt quite a lot from their contemporaries in other countries. Although the concept of building light truss bridges to serve a railway while trade built up was new to Britain in the 1860s and 70s, it had been accepted practice in America for more than 20 years. As a result, engineers in that country met with most of the problems pertaining to such structures before the British came up against them. The following extracts dealing with wind loading come from Hermann Haupt's widely circulated treatise on bridge construction which was published as early as 1856.[27] He wrote, "The use of lateral bracing is principally to guard against the effects of wind, and other disturbing causes tending to produce lateral flexure in the roadway. The ordinary bracing to resist this action consists of ties and braces similarly disposed to those in the main truss except that equal strength is required in the direction of each diagonal of the horizontal panels. The greatest lateral strain is that produced by the action of a high wind; assuming the force of wind at 15 lbs/ft^2 (725 N/m^2) as a maximum ... (he gives a specimen calculation for the particular truss he is discussing) ... the effect of this force would be estimated precisely as the strain of a uniform load upon a bridge ...".

Later, (one of the book's faults is that it is imprecisely laid out and rather rambling - probably due to the fact that it was written over a period of about 10 years) the author was less certain about the allowance of 725 N/m^2. He noted this in discussing the fate of one of his own bridges carried away by a sudden wind:[28] "It is desirable that further experiments should be made to ascertain the force of wind in violent storms. It is probable that it is generally underrated. The writer addressed letters to gentlemen who had been engaged in making observations with the anemometer. The most satisfactory answer was given by Professor Bache who stated that on Saturday 5th August 1843 at 8 o'clock p.m. a tornado passed within a quarter of a mile of the observatory at Girard College and the force of wind was so great as to exceed the range of the spring and to break the wire connecting it with the plate of the anemometer; the force required exceeded 42 lbs to the square foot (2040 N/m^2) which was the range of possible movement of the registering arm. The next greatest force of wind was 14 lbs to the square foot (680 N/m^2) from 4 to 5 o'clock a.m. on the 17th February 1842. From 0. to 5 hours a.m. on the same day the mean was 12.6 pounds (610 N/m^2) from 0 to 11 hours a.m. the mean was 11.4 pounds (550 N/m^2) and for the day 7.69 lbs. (370 N/m^2)."

Here, then, the problem was isolated ten years before the first wind loading failure of a bridge in service, but even in his own country Haupt's words went unheeded; partly, no doubt, because of the poor quality and rather outdated content of much of his text but also because there was no provision for research among the bridge-builders of his day - engineers who fought each other for design and construct contracts had no money to spare for projects which were for the general good.

This is reflected by the effect of the failures listed in Table 3.2. which were rather flippantly used by the American engineers as full

TABLE 3.2

AMERICAN TRUSS BRIDGES DESTROYED BY WIND

Name	Date Wrecked	Description
Havre de Grace Md.	1866	Ten 76 m (250') spans of a wooden Howe truss bridge blown over. Uniform wind load to cause destruction 1310 N/m^2 (27 lbs/ft^2)
Decatur Alabama	1870	Two spans of a truss bridge blown over; force required 1260 N/m^2 (26 lbs/ft^2)
Omaha Nebraska	1877	Two 76 m (250') spans of an iron Post truss blown over; force required 905 N/m^2 (18.7 lbs/ft^2)
Meredosia Illinois	1880	One 46 m (150') span of a wooden Howe truss overturned. Force required 1160 N/m^2 (24 lbs/ft^2)

scale experiments suggesting a lower bound to a design wind pressure arbitrarily assumed to be uniform. Whenever a collapse occurred the various engineers made a slight alteration to the specifications which were then fashionable as a basis for awarding design and construct contracts. By the mid 1870s a typical figure for wind loading was about 2000 N/m2 which was reasonable enough except that it was based on experience rather than experiment.[29]

Just before the Tay Bridge opened T C Clarke came over from America to read a paper and lead a discussion on iron bridges of very long span at the Institution of Civil Engineers.[30] The practice with which he was familiar was by this time sufficiently advanced for him to be shocked by the attitude of English engineers to wind load, and in particular by the fact that they did not recognise that these forces could constitute the limiting factor for bridge design. "A bridge is a complex structure," he said, "because it has to bear not only the forces of gravity but also the side pressure of the wind. Mr Young stated that it is a simple matter to provide against the force of wind but it is really the most difficult and complicated part of the problem. The most economical depth possible has to be used to resist the force of gravity but then the side pressure prevents the use of an economical height; consequently the bridge when finished is a compromise between the results of the two forces. This is why the long span bridges are comparatively not so high as those of shorter span. In spans of less than 200 feet the proportion is 1/5 or 1/6."

This statement confirms the earlier evidence that American practice was considerably ahead of that in Britain during the 1870s, thus proving that British engineers could have learned something useful by keeping track of developments abroad.

3.3.3. Developments in Britain

However, it is more realistic to see what could have been learnt from works carried out at home. This is because the necessary lines of international communication were not established until the mid-1870s when important structures like the Severn and Tay bridges were already under construction.

It is possible that important trends in design and construction could have been detected from a continuous record of the structures being built in Britain. To elaborate, the general direction of bridge building during the hundred years after the beginning of the Industrial revolution (c.1760) was towards structures which used progressively less material to carry a given load (Tables 3.3 and 3.4.). At the beginning of this period there were only two types of crossing: masonry arches where durability and/or appearance were important, and timber trestles for a host of moreorless makeshift structures throughout the country. Wind loads did not seriously affect these bridges, in the case of stone structures because the weight of masonry was adequate to resist overturning and in the case of the wooden bridges because the sites were never particularly exposed. At this time other problems were much more important - scour round the piers and the action of floods being typical examples.

During the last twenty years of the 18th century and the first twenty of the 19th cast iron became accepted as an alternative material for building arch bridges (fig 3.8). Although the weight of these structures was only one or two per cent of a comparable masonry bridge the spans built were never large or exposed enough to be susceptible to wind damage. Anyway the arch bridge which presents little surface area

TABLE 3.3

SOME IMPORTANT BRIDGES IN BRITAIN 1770 - 1830

Type	Name	Date	Designer	Loading R=Road C=Canal	Spans(m)	Span weight (tons)	Deck depth	Deck width	approx. Clearance
Cast Iron Arch	Ironbridge	1779	Pritchard	R	31	378	VARIABLE	7	17
	Sunderland	1796	Paine	R	72	260		9	39
	Pont Cysyllte	1805	Telford	C	19x16	24		3.7 3.7	39
	Southwark	1819	Rennie	R	63-73-63	1665		13	10
	Tewkesbury	1826	Telford	R	52	500		7	9
Stone Arch	Over	1827	Telford	R	46	4000		8	9
Suspension	Union	1820	Brown	R	110	100	0.5	4.5	10
	Menai	1826	Telford	R	168	644	0.5	9	33
	Hammersmith	1827	Clark	R	129	-	1.5	9	10

TABLE 3.4

WROUGHT IRON RAILWAY BRIDGES IN EXPOSED SITUATIONS

Name	Designer	Length of Crossing(m)	Max. clearance (m)	Date of Completion	Number of Tracks	Main span length (m)	span weight (tons)	Deck width(m)	Deck depth (m)	Cost £10^3
Tay	Bouch	3170	27	1878	1	75 (max)	192 288	5	9	350
Victoria (Montreal)	R.Stephenson	2900	12	1859	1	100 (max)		5		1350
Severn	Keeling & Owen	1270	21	1879	1	100 (max)	200		bow/ string	190
Saltash	I.K.Brunel	673	32	1859	1	139		5		225
Boyne	McNeil & Barton	535	27	1855	2	80	362	9	7	140
Kent	Brunlees	480	7	1857	1	9	8	5	1	15
Leven	Brunlees	480	8	1857	1	9	8	4	1	18
Britannia	R.Stephenson	460	37	1850	2	152	1420	4.5 per track	9	600
Crumlin	Kennard	460	63	1857	2	46	88 -	8	5	62
Belah	Bouch	300	60	1861	1	18	16 -	6	2	25

Notes: Where two figures are given for span weight, the upper is the weight of iron

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Fig 3.8. Two 150ft. span bridges by Telford.
Above, the iron arch at Bonar (1811-12)
Below, the masonry arch at Over (1825).

to the wind at mid span is inherently one of the better forms that can be exposed to a gale.

Suspension bridges became popular after 1810, and frequently suffered destruction in high winds. Engineers might have made a note that wind (not just aerodynamic effects) needed consideration in light bridges but they apparently overlooked this. The railway builders can be excused on the grounds that they soon discovered that the suspension bridge as known at the time proved to be wholly inadequate for train loadings. Moreover the first generation (1830-55) of railways in England demanded and were given monumental structures for their principal bridges in which wind forces were insignificant. Cyclones and hurricanes seemed to be the only wind loads to be guarded against but since these freak storms rarely hit the British Isles no special provisions were made.

In the 1860s & 70s railway investors turned their attention to the construction of secondary and in many cases speculative railways, the great main lines having become well established by that time. The new routes were generally built as cheaply as possible so that the promoters would not lose too heavily even if their line failed to generate any traffic, but nevertheless there were frequently formidable physical obstacles to overcome - such as long estuary crossings or high level bridges above deep valleys. Against this background light truss bridgework became an important type of structure, and built by a master engineer it could do its job effectively and efficiently. But in the hands of a second rate designer, an engineer content to apply existing techniques to a new type of bridge, difficulties and deficiences could and did arise.

This situation could probably have been avoided if contemporary practice had included proper provision for reviewing new developments. Such effort as was made in this direction was never published in a more helpful form than the sort of classification presented in tables 3.3 and 3.4. The old collections of data and statistics from which these figures have been taken just tried to give a general picture of a number of structures and thus different authors collected different parameters.[31] As Benjamin Baker commented in his book on long and short span railway bridges[32] this meant that there was really no chance of comparing structures even when they were designed to fulfil the same purpose.

3.3.4 Important Design Parameters

A satisfactory review might have been possible had an attempt first been made to isolate the really important features of contemporary design philosophy. Looking back over the mid-century source material it can be seen that the contemporary mind regarded low cost, economy of material, rigidity and ease of construction as the four most important ingredients of a satisfactory bridge. Of these cost does not constitute a valid basis for comparing structures because, quite apart from the fact that there were great fluctuations in the price of iron in the mid 19th century,[33] the money spent on nominally similar structures was always subject to local variation. High cost resulted from matters such as awkward foundations, an isolated site or the need to work exceptionally quickly. Unusually low estimates sometimes reflected the outstanding efficiency of an individual builder but might equally conceal a skimped programme of works or, possibly, inadequate design.

With regard to other design factors, it is reasonable to say that between 1855 and 1875 the train loads to be provided for in medium span structures did not alter very much[34] (fig 3.9), that the acceptable degree of rigidity remained constant,[35] and that ease of construction was a matter of subtlety in design detail rather than overall design conception.[36] Consequently the comparison of the structural efficiency achieved in the bridges of the period becomes a matter of tabulating the quantity of material used for the various elements of the structures. Of these the most important were the iron piers and the girderwork supported upon them.

The economic weight of material in bridge spans was a subject which interested writers as well as practising engineers. While

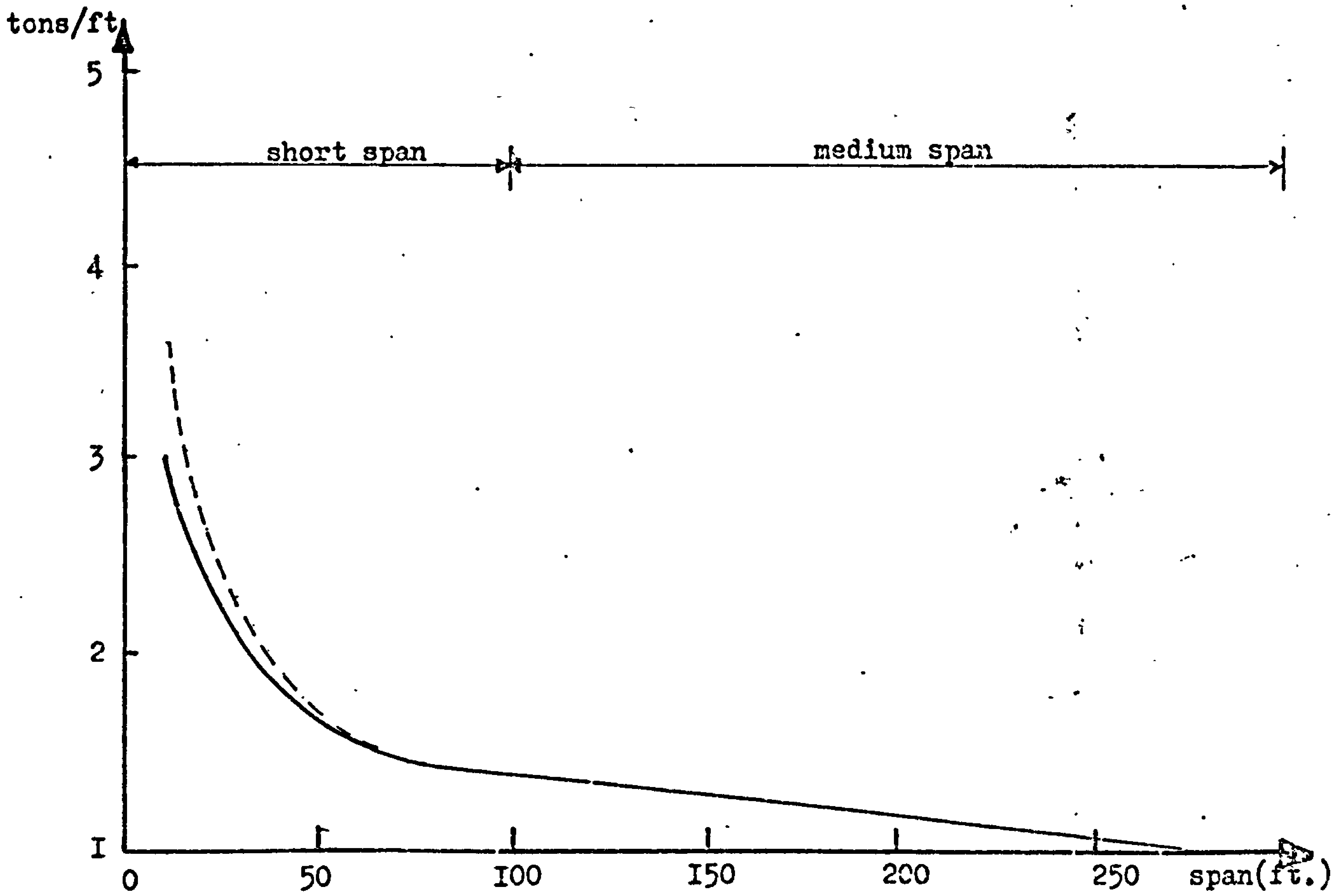

Fig 3.9. Assumed live loads for railway bridges.
Early railway engineers assumed live loads at one ton/ft.
The graph shows information abstracted from B.Baker's "Short span railway bridges"(I873). The basic figures are shown by the unbroken line; the dotted line includes the allowance made for fatigue,jolting,high speed etc.

acknowledging that a mathematical treatment linking the many possible variables could not be attempted, they put forward empirical formulae from time to time. The most interesting of these are represented graphically on fig. 3.10.

Rankine's expression suggested that the weight of metal for a new span could be predicted from the weight used in an existing bridge of comparable design but different span and design loads.[37] A defect of this method was that it gave unrealistically low weights for short bridges.

This was remedied by Benjamin Baker in his essay on Short Span Railway Bridges,[38] where a table of weights based on a combination of theory and the results achieved in practice was given. These figures were based on the most up to date information available regarding railway loads, and can be taken as a satisfactory upper bound for the end of the period. The graph modelled on Rankine's method applied to a successful but outstandingly light bridge provides the locus of a lower limit.

It will be seen that the weight of iron in the main Tay Bridge girders lies between these curves so the conclusion is that these members contained enough material for a satisfactory structure to be formed.

Although the thirteen longest spans were wrecked in the great storm of 1879 many of the remainder were transferred to the new Tay Bridge (1882), thus suggesting that something other than the girders constituted the primary deficiency in the structure.

The other main component of light truss viaducts were the piers

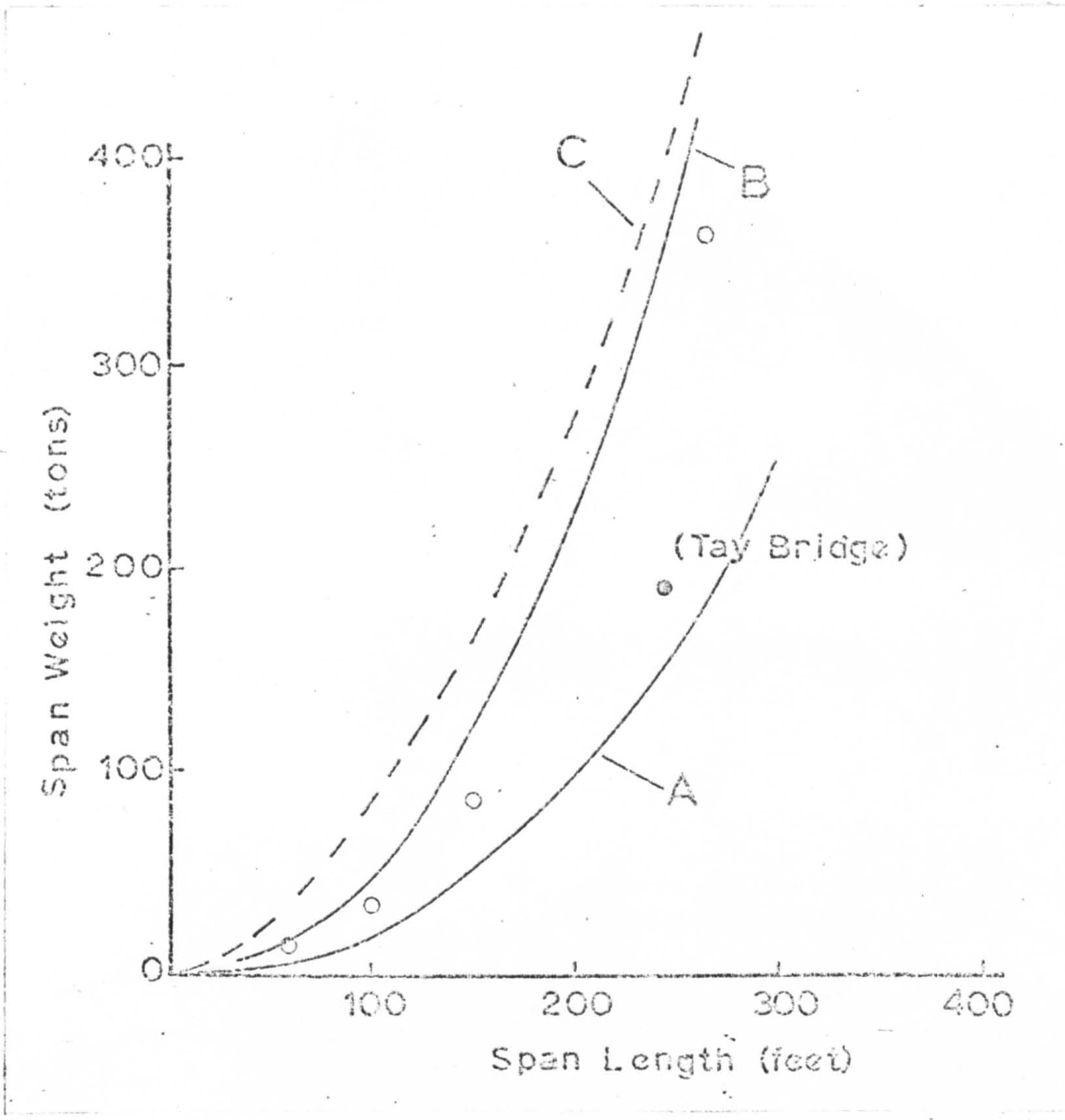

Fig 3.10. The weight of material in bridge spans.

A based on Rankine's formula of 1862, assuming live load one ton/ft., simply supported spans, single track railway, depth/span of girder 1/10, wrought iron construction.

B ditto, for double track railway (i.e. live load of two tons/ft.)

C Baker 1873; double track railway, simply supported spans live load varying as shown in fig 3.9.

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Fig 3.11. Boyne Viaduct.

supporting the spans. In the earliest British examples masonry towers were used (fig. 3.11), but it was soon shown that iron could provide a cheaper solution.[39] Designers found that they had to find a balance between using a few heavily built towers supporting long spans and more, lighter piers carrying shorter trusses. Low level crossings obviously favoured the latter solution while high bridges usually had longer spans.

In the words of Humber's book on bridge construction[40] the "...first point to be considered (in the design of piers) is the area required to support the structure and load. As the weight acts directly upon the piers without experiencing any deviation from its normal direction, the area to support it will vary as the load supported . . .".

The author did not say much more about pier design but even the criterion quoted is sufficient to suggest that the Tay Bridge structure was most unusual in this respect. From the series of diagrams (figs. 3.12 - 3.15: 3.12 - 3.14 are of structures which were recognised as innovatory and daring in their own day), it can be seen that the ratio of span weight to pier weight was very high in the Tay Bridge, and that the piers were very light for a structure crossing a wide estuary, very much lighter for example than the piers built for the Severn Bridge.

Turning to the silhouette drawings (figs. 3.16 - 3.22) the completely unprecedented length and exposure of the bridge can be readily appreciated.

Combining these several observations, it is tempting to suggest that an astute engineer, even if versed solely in British practice, could have detected that the structure was a sufficiently new departure to merit special investigation. Further clues that the bridge was inadequate to resist wind loads could have come from the experiences of engineers in America, (mentioned above), and from a fully comprehensive register of innovatory bridges at home and abroad. Unfortunately for Bouch none of these aids were readily available to British engineers in the 1870s although the source material was all accessible in one form or another.

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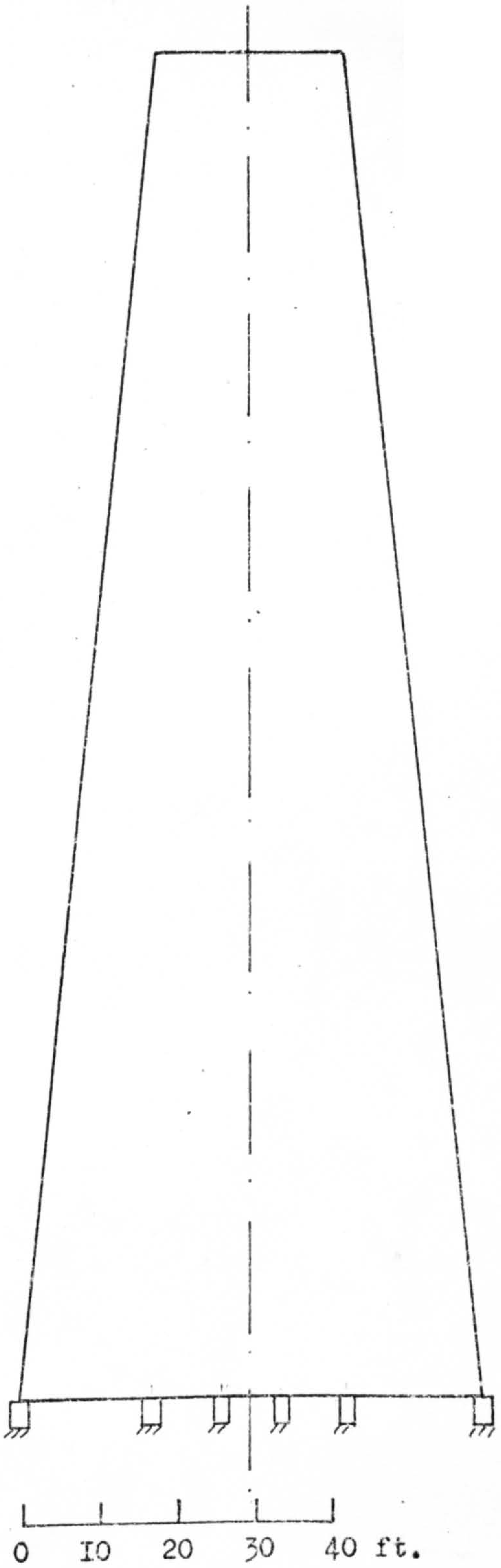

Fig 3.I2. Crumlin Viaduct: the Isabella pier.

Weight of superstructure supported...I00 tons
Weight of pier... 200 tons
Resistance to overturning about base.9000 tons ft.
Span length... I50 ft.

Liddel, Gordon and Kennard, engineers.

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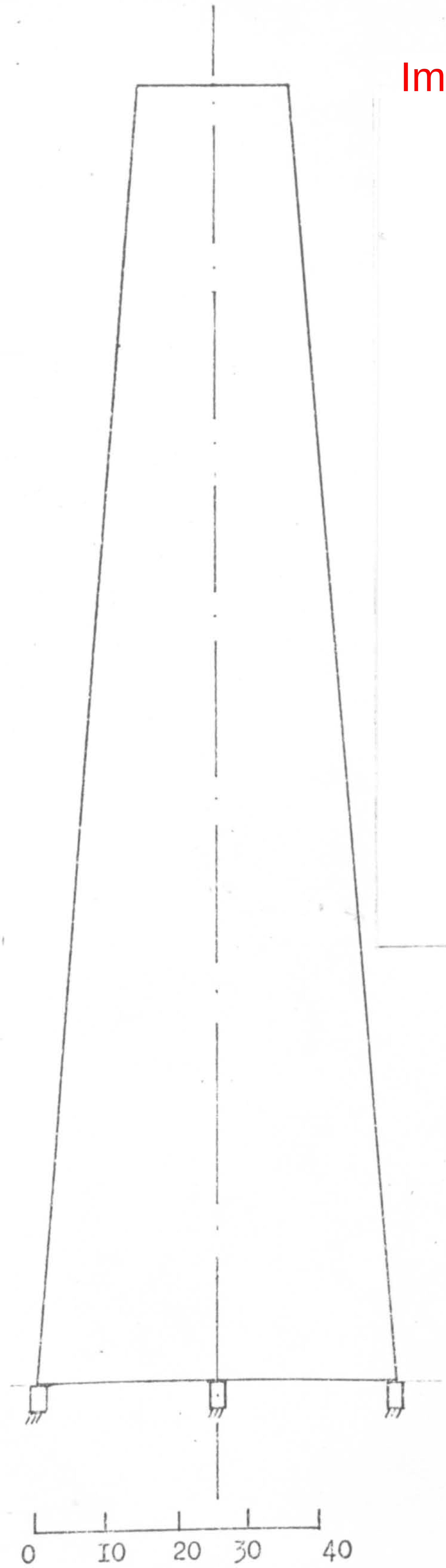

Fig 3.I3 Beelah Viaduct pier

Weight of superstructure supported...20 tons (approx)
Weight of pier...   II5tons
Resistance to overturning about base.3280tons ft.
Span length...   60 ft.

Thomas Bouch, engineer.

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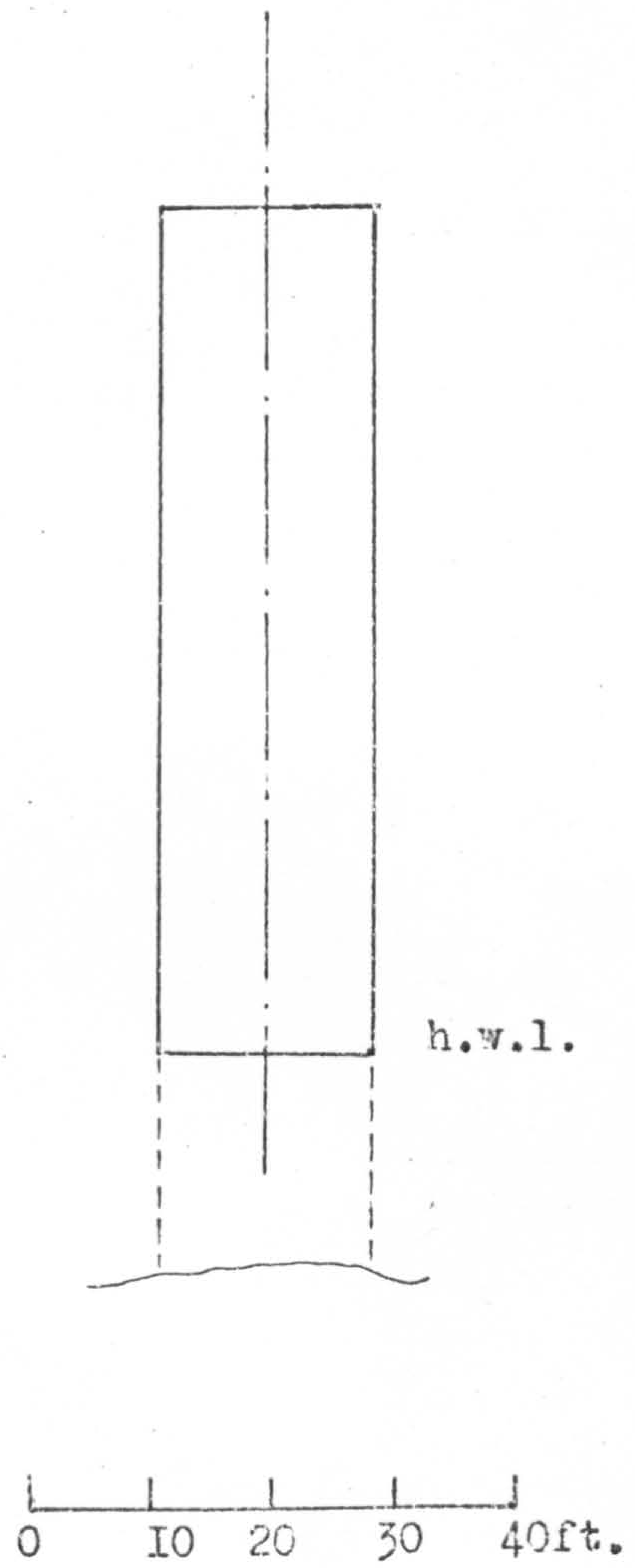

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Fig 3.14
The Severn railway bridge,
pier for a typical 134 ft.
span.

Weight of superstructure supported...	200 tons (approx)
Weight of piers above high water ...	665 tons (approx)
(the cast iron tubes shown above were filled with concrete and braced very heavily from top to bottom).	
Resistance to overturning about base.	7800 tons ft.
max span length	327 ft.

Keeling and Owen, engineers.

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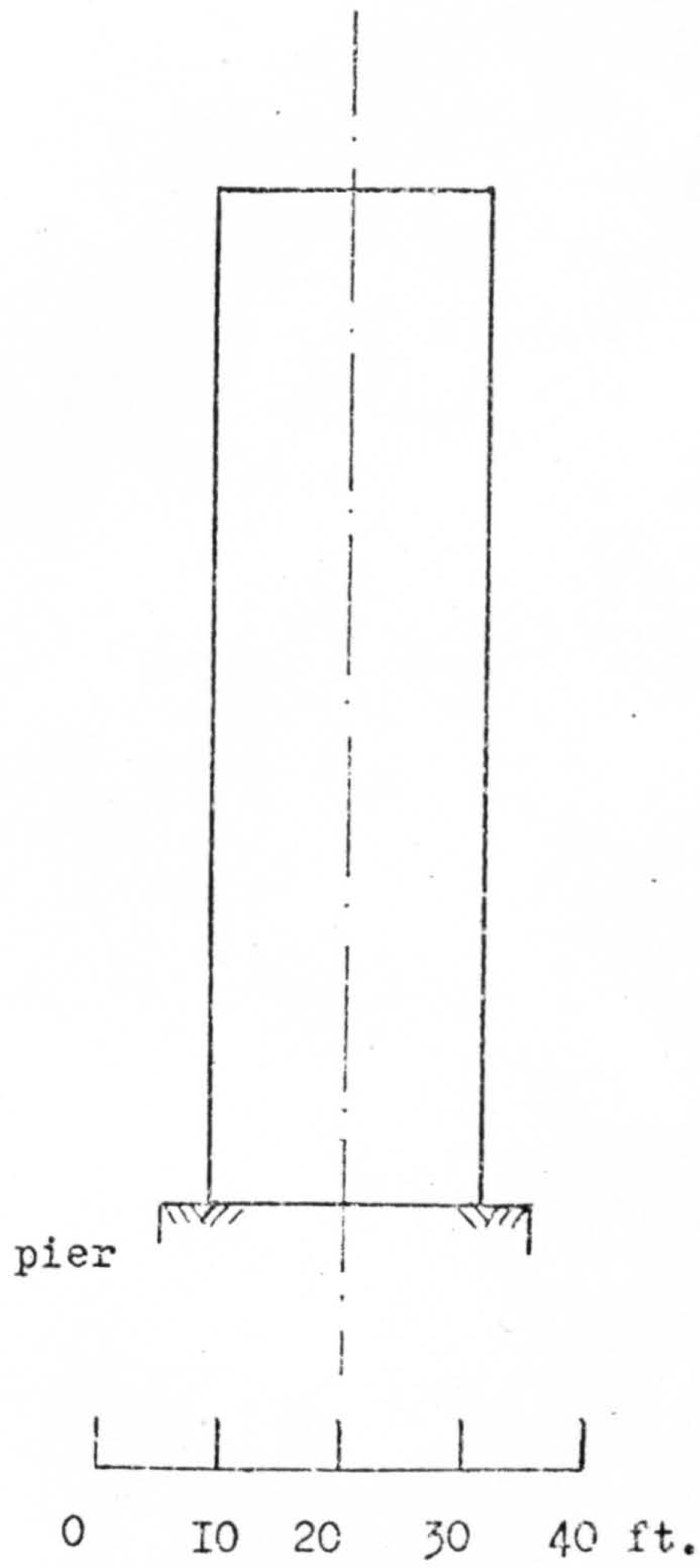

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Fig 3.I5.
The Tay railway bridge.
pier for a typical high
girder span.

Thomas Bouch, engineer.

Weight of superstructure supported...	288 tons
Weight of iron pier (main tubes filled with concrete). ...	I05 tons
Resistance to overturning about base.	4320tons ft.
Max span length	245 ft.

COMPARATIVE ELEVATIONS OF BRIDGES.

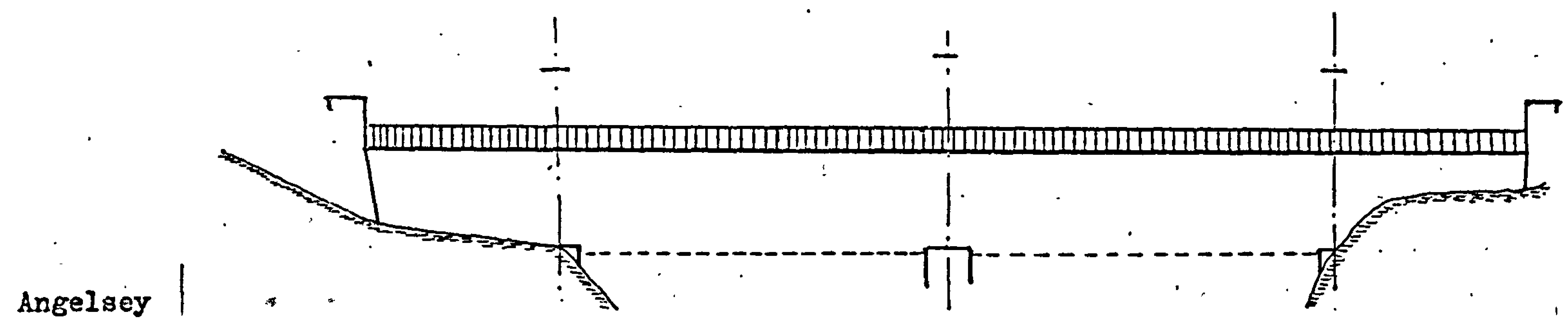


Fig 3.16. Britannia bridge.

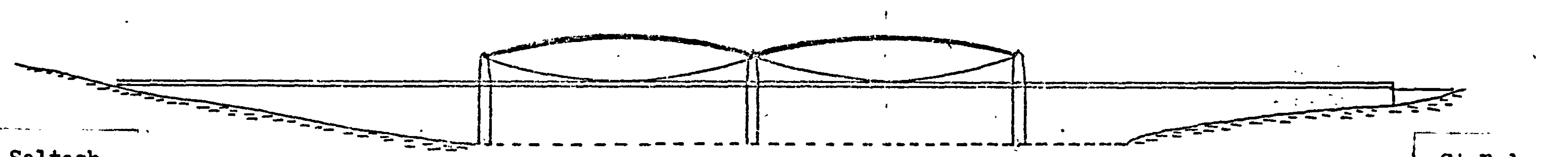
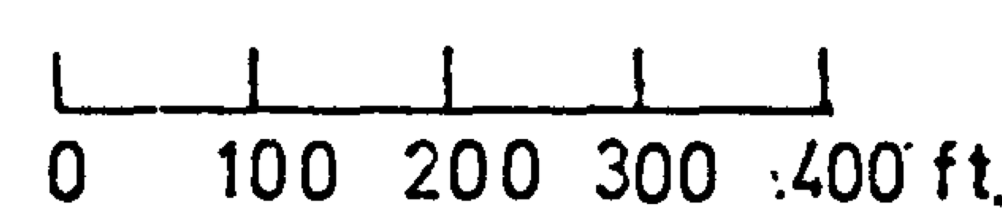


Fig 3.17. Royal Albert Bridge, Saltash: cross-section of bridge site along axis of main spans



COMPARATIVE ELEVATIONS OF BRIDGES.

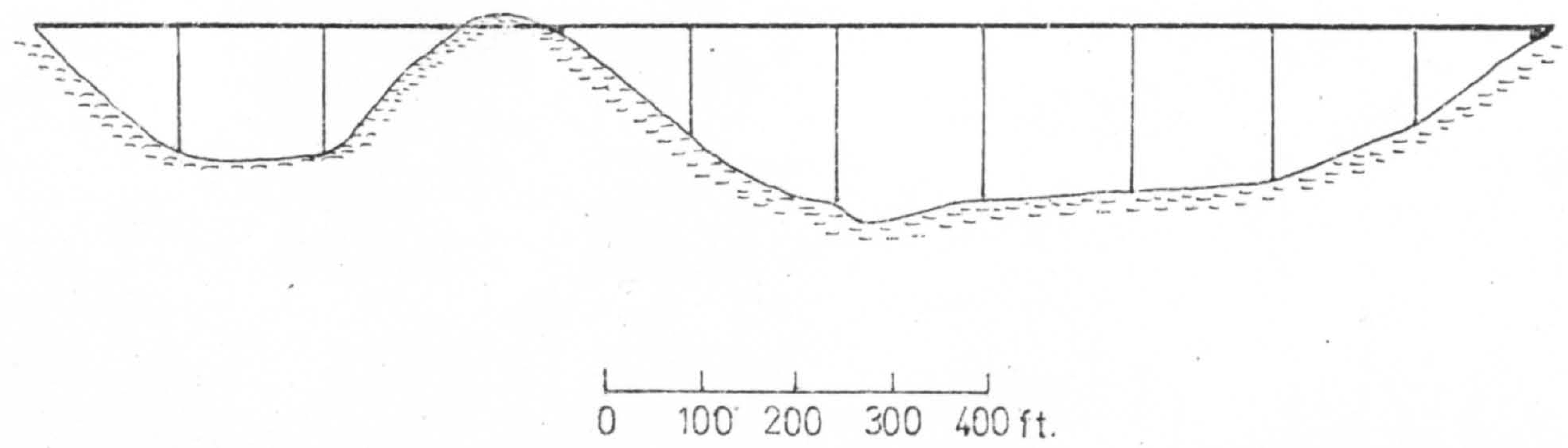


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Fig 3.18. Crumlin Viaduct.

COMPARATIVE ELEVATIONS OF BRIDGES.

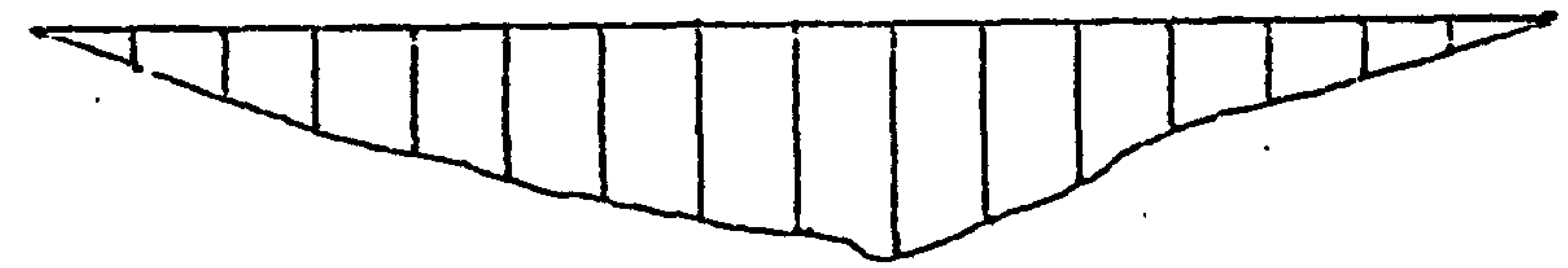
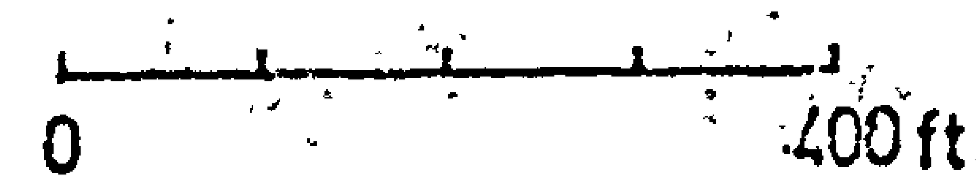


Fig 3.19. Belah Viaduct.

Fig 3.20. Leven Viaduct.



COMPARATIVE ELEVATIONS OF BRIDGES.

Fig 3.2I. The Tay railway bridge, showing 3,000ft of the high girder section of the IO,300ft. structure.

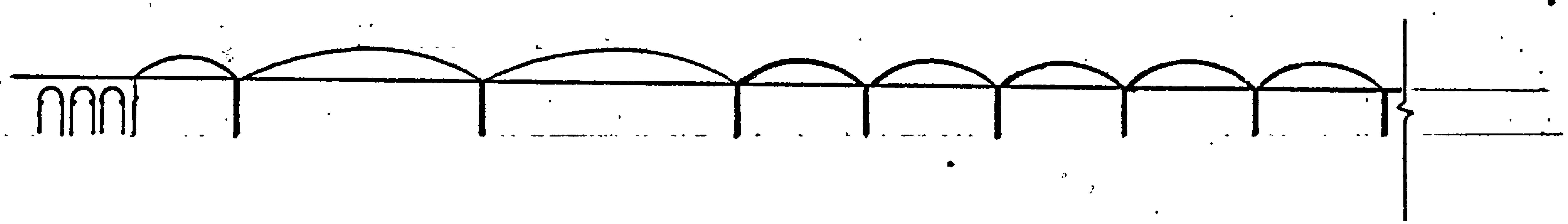


Fig 3.22. The Severn railway bridge opened in I879, nearly a year after the Tay bridge. This silhouette shows the more exposed Welsh half of the bridge.

3.4. Summary

Some conclusions relevant to modern practice are presented to close this survey of the Tay Bridge tragedy. The first part of the discussion of the wind loading problem showed how Bouch managed to build an immense bridge using invalid design information, and pointed out that the Astronomer Royal should never have given advice in such a form that the uncertainties about the magnitude and duration of wind forces were glossed over.

A parallel relevant to present day engineering can be drawn from the modern use of Codes of Practice for design. All the information contained in these documents should be backed up by a handbook covering the data on which they are based and the range of applications the drafting Committee had in mind when the document was prepared. At the present time some, but not all, codes are supported in this way.

The second section of the discussion suggested that there is a point in trying to document and review developments in various aspects of Civil Engineering procedure as they take place.

In the case of the Tay Bridge a properly kept record of changes in design parameters which the contemporary engineers considered important would have helped avert the tragedy. Classified information of this type should be maintained today to enable a watch to be kept on the working of Codes of Practice.

It was also shown that a register of accidents would have shown the susceptibility of light trusses to wind forces several years before Bouch's bridge was destroyed. This suggests that it is worth keeping a careful record of all incompletely explained accidents with a view to detecting new problems before they cause a major collapse.